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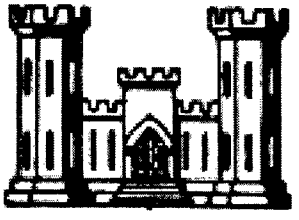
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30 November 1970

Change 2 dated 20 August 1986
has been inserted.

ENGINEERING AND DESIGN

LABORATORY SOILS TESTING



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HEADQUARTERS, DEPARTMENT OF THE ARMY
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-Foreword

This manual was prepared at the U. S. Army Engineer Waterways Experiment Station (WES) under direction from the Office, Chief of Engineers (OCE). General supervision and technical guidance were provided by R. A. Barron, Chief, Soil Mechanics Branch, Engineering Division, OCE, and M. M. Johnston and R. R. W. Beene of that Branch.

The manual was prepared by the staff of the Embankment and Foundation Branch, Soils Division, WES; the principal authors were B. N. MacIver and G. P. Hale.

DEPARTMENT OF TEE ARMY
Office of the Chief of Engineers
Washington, D.C. 20314

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Change 1

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1 May 1980

Engineering and Design
LABORATORY SOILS TESTING

1. This change to EM 1110-2-1906, 30 November 1970, provides the updating of the TABLE OF CONTENTS and the addition of the following four (4) APPENDICES:

a. Appendix VIII A; Swell and Swell Pressure Tests

b. Appendix XA: Cyclic Triaxial Tests

c. Appendix XB: Determination of Critical Void Ratio

d. Appendix XIII: Pinhole Erosion Test for Identification of Dispersive Clays

2. Substitute the attached pages as shown below:

Remove pages	Insert new pages
i, ii, iii, iv, v, vi, and vii	i, ii, iii, iv, v, vi, vii, viii, ix, x, xi, xii, and xiii

3. Add Appendices VIII A, XA, XB, and XIII.

4. File this change sheet in front of the publication for future reference purposes.

FOR THE CHIEF OF ENGINEERS:

Forrest T. Gay

FORREST T. GAY, III
Colonel, Corps of Engineers
Executive Director, Engineer Staff

THIS CONSTITUTES CHANGE 1 TO EM 1110-2-1906, 30 NOV 70

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DEPARTMENT OF THE ARMY
U. S. Army Corps of Engineers
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1 May 80

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ENGINEERING AND DESIGN

LABORATORY SOILS TESTING

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	A	Area
	C_c	Compression index
	c	Cohesion or cohesion intercept of strength envelope based on total stresses
	c'	Cohesion intercept of strength envelope based on effective stresses
	D	Diameter
*	D_1,D_2	Dispersive soil classification--pinhole erosion test
	D_{10}	Diameter of soil particle having a size greater than 10 percent (by weight) of the particles
	D_d	Relative density
*	D_i	Initial diameter of pinhole
	e	Void ratio
*	e_c	Void ratio after consolidation
	F	Applied shear force
	G_a	Apparent specific gravity
	G_m	Bulk specific gravity
	G_s	Specific gravity of solids
	H	Henry's coefficient of solubility
	H	Height
*	H_i	Initial hydraulic head in pinhole erosion test
	ΔH	Change in height

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* † Omitted here are the symbols A , D , H , S , V , and w having the subscripts o , c , and f , and the symbol e having the subscripts o and f (for example, A_o , H_c , w_f , etc.); these subscripts indicate that the symbols refer to properties of test specimens measured or computed under the following conditions: *

o	Initial or before test
c	After consolidation or before shear
f	Final or after test

	ΔH_o	Change in height during consolidation or before shear
	h	Hydraulic head
	i	Hydraulic gradient
	k	Coefficient of permeability
	L	Length
	L_s	Linear shrinkage
	LL	Liquid limit
	N	Number of blows of liquid limit cup
*	ND_1,ND_2	Nondispersive soil classification--pinhole erosion test
*	ND_3,ND_4	Intermediate soil classification--pinhole erosion test
	n	Porosity
	P	Applied axial load
	PI	Plasticity index
	PL	Plastic limit
	p	Pressure
	pc	Preconsolidation pressure
	p_o	Overburden pressure
	Q	Quantity of flow
	q	Rate of discharge
	q_u	Unconfined compressive strength
	R	Shrinkage ratio
	S	Degree of saturation
*	SAR	Sodium absorption ratio
	SL	Shrinkage limit
	s	Shear strength
	s_u	Undrained shear strength
	t	Time
	t_{50}	Time to 50 percent primary consolidation

	t_f	Time to failure
	U	Degree of consolidation
	u_a	Atmospheric pressure (absolute)
	u_o	Required back pressure (gage)
	V	Volume
	V_a	Volume of air
	V_s	Volume of soil solids
	V_v	Volume of voids
	V_w	Volume of water
	ΔV	Change in volume
	ΔV_w	Change in volume of water
	W	Weight of wet soil
	W_s	Weight of dry soil
	W_w	Weight of water
	w	Water content
	w_N	Water content at N blows
	β	Slope angle of flow line
	γ_d	Dry unit weight (or dry density) of soil
	γ_m	Wet unit weight (or wet density) of soil
	γ_w	Unit weight of water
	ε	Axial strain
*	ε_{da}	Double amplitude axial strain
	η	Coefficient of viscosity
	σ	Normal stress
	σ_1	Major principal stress
	σ_3	Minor principal stress
	$\sigma_1 - \sigma_3$	Deviator stress
*	σ_{dc}	Cyclic deviator stress
*	$\frac{\pm \sigma_{dc}}{2 \sigma_{3c}}$	Cyclic stress ratio

*	σ_{3f}	Effective confining pressure at failure
*	σ_{fc}	Consolidation stress on the failure plane
	τ	Shear stress
*	τ_{cyclic}	Cyclic shear stress
	ϕ	Angle of internal friction (or slope angle of strength envelope) based on total stresses
	ϕ'	Angle of internal friction (or slope angle of strength envelope) based on effective stresses
	ϕ'_r	Angle of friction determined from the drained repeated direct shear test

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ENGINEERING AND DESIGN

LABORATORY SOILS TESTING

1. PURPOSE. This manual presents recommended testing procedures for making determinations of the soil properties to be used in the design of civil works projects. It is not intended to be a text book on soils testing or to supplant the judgment of design engineers in specifying procedures to satisfy the requirements of a particular project.

2. APPLICABILITY. The provisions of this manual are applicable to all divisions and districts having Civil Works functions.

3. NOTATIONS, A partial list of the symbols used in this manual appears following the Table of Contents. Not included in the list are symbols used arbitrarily in formulas to facilate computations and those having a special meaning within a particular appendix. All symbols are defined where they first appear in an appendix and wherever restatement may be needed for the sake of clarity. Whenever possible, these symbols correspond to those recommended by the Committee on Glossary of Terms and Definitions in Soil Mechanics of the Soil Mechanics and Foundations Division of the American Society of Civil Engineers.†

4. REFERENCES. The material presented in this manual has been drawn from many sources, persons, and organizations; wherever possible, specific references are given by footnotes. In general, the procedures and practices herein have been taken, under the guidance of the

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† "Glossary of terms and definitions in soil mechanics," Proceedings, ASCE, vol. 84, No, SM4 (October 1958).

Office, Chief of Engineers, from the experience of the U. S. Army Engineer Waterways Experiment Station and the U. S. Army Engineer Divisions. Further contributions have come from Harvard University, the Massachusetts Institute of Technology, the U. S. Bureau of Reclamation, and the American Society for Testing and Materials.

5. TESTING PROCEDURES. Since soils exist in an enormous variety, and since the problems of applied soil mechanics also exist in a very great variety, testing procedures for determining the engineering properties of soils (such as strength-deformation relationships) must not, in fact, cannot, be standardized. Before any soils testing is requested of a laboratory, the design engineer responsible for formulating the testing program must clearly define the purpose of each test to himself and to the person who will supervise the testing.

It is generally necessary to adapt the testing procedures to the specific requirements of an investigation. For example, the consolidation test can be performed in various ways. What is often called the " standard consolidation test" is performed by always doubling the previous load on the specimen. This procedure will produce tin-&-consolidation curves that usually permit the most precise evaluation of the coefficients of permeability and consolidation. However, these load increments are not always satisfactory for defining the preconsolidation pressure from the shape of the void ratio-pressure curve; for this purpose a much smaller factor than 2.0 should be used during incremental loading. Also, the maximum load to which a consolidation test should be continued will depend on the consistency and stress history of the soil and the requirements of the project. For example, if a clay which had been normally consolidated under an effective overburden pressure of 0.5 ton per sq ft is to be loaded by an embankment which will exert an additional pressure of 1.0 ton per sq ft, the consolidation test need not be continued beyond a load of 4.0 tons per sq ft to fulfill the purpose of the test. On the other hand, a highly overconsolidated clay which will be loaded by an embankment of substantial height may require that the consolidation test be continued to a loading

of 20 tons per sq ft or more.

An even greater variety in testing procedures exists for measuring the strength of soils, and the purpose of the tests must be constantly reviewed to insure that the results have meaning with respect to design.

Tests which do not measure clearly defined engineering properties (such as Atterberg limits, specific gravity, grain-size analyses, and compaction), however, do require adherence to standardized procedures. Even here the dangers of injudicious testing must be recognized As an example, compaction test results-must be carefully evaluated if the material coarser than 3/4 in. (or some other size) has been removed according to the standard method.

Procedures for soils tests necessary for the design of Civil Works projects appear as appendixes to this manual. The procedures are considered to represent the best current guidance for obtaining acceptable de sign data, Deviations from these procedures may be necessary on occasion, according to the judgment of testing or design engineers, their experience with local soils, or peculiarities of a project. However, to insure that the test methods remain compatible with the purpose of the tests and that the results will be acceptable, every such deviation should be discussed in advance with the design office requesting the tests or, if in the judgment of the design engineer such deviation represents a major departure from the conventional procedure, should be first approved by the Office, Chief of Engineers. Also, a description of any non-conventional procedure must accompany the test data.

6. RELIABILITY OF TESTING APPARATUS AND RESPONSIBILITY OF PERSONNEL, All who are engaged in soils testing must constantly be aware of the importance of accuracy in measurements. Inaccurate measurements will produce test results which are not only valueless but are misleading. Each appendix to this manual contains a list of the more common possible errors associated with the procedures described in that appendix. Serious errors can be caused by poorly constructed apparatus (for

example piston friction in triaxial compression chambers or rough-finished consolidometer rings), by maladjusted apparatus (liquid limit devices, proving rings, or mechanical compactors), and by worn parts (liquid limit cup or grooving tool or knife edges of lever systems). Regular calibration and inspection must be a standard practice in all laboratories.

The personnel performing the tests must be thoroughly familiar with the apparatus, the testing procedures, and good laboratory technique in general. They must be conscientious in the handling of soils and must appreciate the purpose of each test they perform. Neat, thoughtful work, with the recording of all test data and a continuous watchfulness for irregularities can prevent most errors. The philosophy should be that one good test is not only far better than many poor tests, but is also less expensive and less likely to permit a misjudgment in design.

7. LABORATORY FACILITIES. A laboratory preferably should be on a ground floor or basement with a solid floor and should be free of traffic and machinery vibrations. Separate areas should be designated for dust-producing activities such as sieve analyses and sample processing. Temperature control of the entire laboratory is to be preferred. If the temperature-controlled space is limited, this space should be used for triaxial compression, consolidation, and permeability testing. A humid room large enough to permit the storage of samples and the preparation of test specimens should be available.

8. SAMPLE HANDLING AND STORAGE. The identification markings of all samples should be verified immediately upon their receipt at the laboratory, and an inventory of the samples received should be maintained. Samples should be examined and tested as soon as possible after receipt; however, it is often necessary to store samples for several days or even weeks to complete a large testing program. Every care must be taken to protect undisturbed samples against damage or changes in water content Such samples should be stored in a humid room and may require rewaxing and relabeling before storage. Except for special purposes,

such as for viewing by designers or contractors or for research, soil samples should not be retained for long periods; even the most careful sealing and storing of undisturbed samples cannot prevent the physical and chemical changes which, in time, would invalidate any subsequent determinations of their engineering properties.

9. SELECTION AND PREPARATION OF TEST SPECIMENS. Under the most favorable circumstances, a laboratory determination of the engineering properties of a small specimen of undisturbed soil gives but an approximate guide to the behavior of an extensive nonhomogeneous geological formation under the complex system of stresses induced by the construction of an embankment or other structure; under the worst circumstances such a determination may have no meaning. Also, the strength, compressibility, and permeability of a soil in place may vary severalfold within a few inches. No other aspect of laboratory soils testing is as important as the selection of test specimens to best represent those features of a foundation soil which influence the design of a project. The selection cannot be based on boring logs alone, but requires personal inspection of the samples and the closest teamwork of the laboratory personnel and the design engineer. This cooperation must be continued throughout the testing program since, as quantitative data become available, changes in the initial allocation of samples or the securing of additional samples may be necessary.

Second in importance only to the selection of the most representative undisturbed material is the preparation and handling of the test specimens to preserve in every way possible the natural structure and water content of the material. Indifferent handling of undisturbed soils can result in test data that are erroneous by several times any errors caused by faulty testing apparatus. With but few exceptions, test specimens should always be prepared in a humid room. Trimming instruments should be sharp and clean and the specimens should be adequately supported at all times; details of the preparation equipment and procedures are presented in the appendixes to this manual. What cannot be gained from any manual,

however, is the judgment and awareness necessary to adjust the techniques for each type of material in order to secure the most satisfactory specimens.

During the preparation of specimens, the laboratory personnel have the best opportunity to record a complete description of the material and to judge whether the material is truly undisturbed. The description should include an identification of the material, its color and consistency, the brittleness of the material and the loss of strength upon remolding, and any heterogeneity or unusual characteristics which might prove valuable in analyzing the test results. Also, any indication of disturbance of boring samples (strata deformed at periphery or distortions concentric with axis of sample) must be noted. Often these distortions cannot be seen except by slowly drying a slice of the material to a water content at which the differences between strata show clearly.+ Photographs of such partially dried slices may be helpful when evaluating the test data and can contribute to improvements in sampling equipment and techniques. Disturbed samples should never be used for any tests other than classification, specific gravity or water content.

10. DATA SHEETS AND REPORT FORMS. Examples of suggested form sheets for recording and computing test data are presented in the appendixes hereto, and some appendixes show the forms to be used for reporting test results. The data sheets shown may be satisfactory in many instances, though each laboratory should adopt whatever data sheets are most suitable for their * practices and apparatus. ENG Form 2086 should be used for presentation of a summary of soil test data in design documents. Well-planned data sheets * can improve the efficiency of testing and, by encouraging the recording of data which otherwise might be lost, can lead to better testing.

The report forms shown in the appendixes have been developed to facilitate the review of design memoranda of Civil Works projects.

+ M. J. Hvorslev, Subsurface Exploration and Sampling of Soils for Civil Engineering Purposes, U. S. Army Engineer Waterways Experiment Station, CE (Vicksburg, Miss., November 1949).

Because they are intended for review purposes, these forms often do not display the test results in sufficient detail for interpretation by the design engineer. Therefore, each laboratory should include with the standard report forms whatever tabulated or plotted data are necessary to satisfy the purpose of a testing program. Graphs should show all the plotted points, not just smooth curves, and be given scales in easily read units, such as 1, 2, or 5 divisions per unit. The report form should contain a complete description of the material, not just the classification, and sketches to illustrate the mode of failure of strength test specimens.

FOR THE CHIEF OF ENGINEERS:



RICHARD F. McADOO
Colonel, Corps of Engineers
Executive

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APP XI	- Unconfined Compr e s sion Test
APP XII	- Relative Density
APP XIIA	- Modified Providence Vibrated Density Test

APPENDIX I:

WATER CONTENT - GENERAL

1. DEFINITION. Water content, **w**, is defined as the ratio, expressed as a percentage, of the weight of water in a given soil mass to the weight of solid particles.

2. APPARATUS The apparatus should consist of the following:

a. Oven, preferably of the forced-draft type automatically controlled to maintain a uniform temperature of 110 ± 5 C throughout the oven.

b. Balances, sensitive to 0.01 g for samples weighing less than 50 g; 0.1 g for samples weighing 50 to 500 g; 1.0 g for samples weighing over 500 g.

c. Specimen Containers. Seamless metal containers with lids are recommended. The containers should be of a metal resistant to corrosion (aluminum is satisfactory). They should be as small and light in weight as practicable in relation to the amount of material to be used in the determination. For routine water content determinations in which specimens weighing between 100 and 200 g are used, a 2-in.-high by 3-1/2-in. diameter container is adequate.

3. SPECIMEN. The amount of material used in the water content determination will generally depend on the maximum size of particles, the amount of material available, and the requirement that the specimen be representative of the material for which the determination is made. When the water is not uniformly distributed throughout the sample, larger specimens will be needed than would otherwise be required. For routine water content determinations on material passing a No. 4 sieve, specimens weighing between 100 and 200 g are adequate, A minimum specimen weight of 500 g is recommended for material having a maximum particle size in the range of the No. 4 to 3/4-in. sieves, and a minimum specimen

weight of 1000 g is recommended for material having a maximum particle size in the range of the 3/4-in. to 1-1/2-in. sieves. Specific amounts of material are required for water content determinations for other laboratory tests; the test procedures should be consulted to determine the proper amounts.

4. PROCEDURE. The procedure shall consist of the following steps:

a. Record all identifying information for the specimen, such as project, boring number, sample number, or other pertinent data, on a data sheet (Plate I-i is a suggested form).

b. Record the number and tare weight of the specimen container.

c. Place the specimen in the container, set the lid securely in position and immediately determine the weight of the container and wet soil by weighing on an appropriate balance.

d. Before the specimen is placed in the oven, remove the lid; the lid is usually placed under the container in the oven. Then place the specimen and container in the oven heated to 110±5C.† Leave the specimen in the oven until it has dried to a constant weight. The time required for drying will vary depending on the type of soil, size of specimen, oven type and capacity, and other factors. The influence of these factors generally can be established by good judgment, and experience with the soils being tested and the equipment available in the laboratory. When in doubt, reweigh the oven-dried specimens at periodic intervals to establish the minimum drying time required to attain a constant weight. For routine water content determinations, specimens consisting of clean sands and

† Laboratory oven drying at 110 C does not result in reliable water content values for soils containing gypsum or significant amounts of organic material. Reliable water content values for these soils can be obtained by drying in oven at 60 Ç±, or by vacuum desiccation. See: U. S. Army Engineer Waterways Experiment Station, CE, A Study of Moisture-Content Determinations on Selected Soils, Miscellaneous Paper No. 4-73 (Vicksburg, Miss., September 1954).

gravels should be oven-dried for a minimum of 4 hr. For most other soils a minimum drying time of 16 hr is adequate, Dry soil may absorb moisture from wet specimens; therefore, any dried specimens must be removed before wet specimens are placed in the oven.

e. After the specimen has dried to constant weight, remove the container from the oven and replace the lid. Allow the specimen to cool until the container can be handled comfortably with bare hands. If the specimen cannot be weighed immediately after cooling it should be placed in a desiccator; if a sample is left in the open air for a considerable length of time it will absorb moisture.

f. After the specimen has cooled, determine its dry weight and record it on the data sheet.

5. COMPUTATIONS. The following quantities are obtained by direct weighing:

a. Weight of tare plus wet soil, g

b. Weight of tare plus dry soil, g

The water content in percent of oven-dry weight of the soil is equal to:

$$\frac{\text{(weight of tare plus wet soil) - (weight of tare plus dry soil)}}{\text{(weight of tare plus dry soil) - (tare)}} \times 100$$

or

$$w = \frac{W_w}{W_s} \times 100$$

where w = water content, percent

W_w = weight of water, g

W_s = weight of dry soil, g

6. POSSIBLE ERRORS.. Following are possible errors that would cause inaccurate determinations of water content:

a. Specimen not representative. The specimen must be representative of the sample as required for the purpose of the determination. For example, a stratified soil may have a great variation in water content between adjacent strata; were it intended to evaluate the strength of the soil on the basis of water content, a large specimen that included material from several strata would not be representative of the weakest stratum. As another example, to determine the average water content of a gravelly clay, the specimen must be large enough to contain representative amounts of both coarse and fine fractions.

b. Specimen too small. As a rule, the larger the specimen, the more accurate the determination because of the larger weights involved.

c. Loss of moisture before weighing wet specimen. Even in a covered container a specimen can lose a significant amount of water unless weighed within a short period.

d. Incorrect temperature of oven. The oven-dry weight of many soils is dependent on the temperature of the oven, so variations in temperature throughout the interior of an oven can cause large variations in the computed water content.†

e. Specimen removed from oven before obtaining a constant oven-dry weight.

f. Gain of moisture before weighing oven-dry specimen.

g. Weighing oven-dry specimen while still hot. The accuracy of a sensitive balance may be affected by a hot specimen container.

h. Incorrect tare weight. The weights of specimen containers should be checked periodically and should be scratched on the containers to avoid possible errors in reading such weights from lists.

---

† T. W. Lambe, Soil Testing for Engineers, John Wiley & Sons, Inc. (New York, 1951).

WATER CONTENT - GENERAL

DATE _____

PROJECT _____

BORING NO. _____

Sample or Specimen No.								
Tare No.								
Weight in grams	Tam plus wet soil							
	Tare plus dry soil							
	Water	W_w						
	Tare							
	Dry soil	W_s						
Water content		w	%	%	%	%	%	%

Sample or Specimen No.								
Tam No.								
Weight in grams	Tare plus wet soil							
	Tare plus dry soil							
	Water	W_w						
	Tare							
	Dry soil	W_s						
Water content		w	%	%	%	%	%	%

Sample or Specimen No.								
Tare No.								
Weight in grams	Tare plus wet soil							
	Tare plus dry soil							
	Water	W_w						
	Tare							
	Dry soil	W_s						
Water content		w	%	%	%	%	%	%

$$w\% = \frac{\text{(tare plus wet soil)} - \text{(tare plus dry soil}}{\text{(tare plus dry soil)} - \text{(tare)}} \times 100 = \frac{W_w}{W_s} \times 100$$

Remarks _____

Technician _____ Computed by _____ Checked by _____

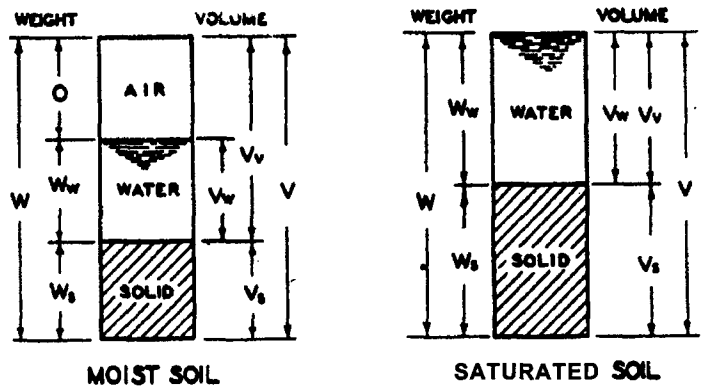
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PLATE I-1

APPENDIX II:

UNIT WEIGHTS, VOID RATIO, POROSITY, AND DEGREE OF SATURATION

1. RELATIONS. A soil mass is considered to consist of solid particles enclosing voids of varying sizes. The voids may be filled with air, water, or both. The fundamental relations of the weights and volumes of the various components of a soil mass can be derived using the simplified sketches shown in Figure 1. Some of the more important relations used in soils engineering calculations are unit weights, void ratio, porosity, and degree of saturation. The quantities which must be known to compute these relations are the weight and volume of the wet specimen, the weight of the same specimen after oven-drying, and the specific gravity of the solids (see Appendix IV, SPECIFIC GRAVITY). The weights of the specimens usually can be obtained without difficulty. The volume of the wet specimen is determined by linear measurement (volumetric method), or by measurements of the volume



WATER CONTENT $w = \frac{W_w}{W_s}$

DRY UNIT WEIGHT (DRY DENSITY) $\gamma_d = \frac{W_s}{V}$

WET UNIT WEIGHT (WET DENSITY) $\gamma_m = \frac{W}{V}$

VOID RATIO $e = \frac{V_v}{V_s}$

POROSITY $n = \frac{V_v}{V} = \frac{e}{1+e}$

DEGREE OF SATURATION $S = \frac{V_w}{V_v}$

SPECIFIC GRAVITY OF SOLIDS $G_s = \frac{W_s}{V_s \cdot \gamma_w}$

NOTE: UNIT WEIGHT OF WATER, γ_w = 62.4 LB/CU FT.

Figure 1. Fundamental relations

or weight of water displaced by the specimen (displacement method). Definitions of the relations to be determined and detailed procedures for determining these values, using the volumetric and displacement methods, are given in the following paragraphs.

2. DEFINITIONS. The unit weights, void ratio, porosity, and degree of saturation are defined below.

a. Dry unit weight, γ_d, or dry density, is the weight of oven-dried soil solids per unit of total volume of soil mass, and is usually expressed in pounds per cubic foot.

b. Wet unit weight, γ_m, or wet density, is the weight (solids plus water) per unit of total volume of soil mass, irrespective of the degree of saturation (see e below). The wet unit weight is usually expressed in pounds per cubic foot.

c. Void ratio, e, is the ratio of the volume of voids to the volume of solid particles in a given soil mass.

d. Porosity, n, is the ratio (usually expressed as a percentage) of the volume of voids of a given soil mass to the total volume of the soil mass.

e. Degree of saturation, S, is the ratio (expressed as a percentage) of the volume of water in a given soil mass to the total volume of voids.

3. VOLUMETRIC METHOD. a. Description. The volumetric method consists of computing the total volume of soil from linear measurements of a regularly shaped mass. In general, the method is applied to soils which can be cut or formed into a cylinder or parallelepiped. Specimens of this type are used for other laboratory tests, and methods for preparing them are described under the individual test procedures. The procedure presented below is based on obtaining a cylindrical specimen by progressive trimming in front of a calibrated ring-shaped specimen cutter. However, other methods for obtaining a regularly shaped mass, such as cutting and trimming or punching, can often be used successfully. The

volumetric method should not be used for soils containing gravel, shells, or foreign materials which would interfere with advance trimming. The calibrated specimen cutter method is particularly suitable for obtaining volumes of silty and sandy soils having little cohesion.

b. Apparatus. The apparatus should consist of the following:

(1) Calibrated ring-shaped specimen cutter, hereinafter referred to as a volumetric cylinder. Types and sizes of volumetric cylinders may vary widely; two types are shown in Figure 2. General requirements are that a volumetric cylinder be made of materials not susceptible of rapid corrosion and that it be as large as possible in relation to the samples being tested. The inside of the cylinder should be polished to a smooth finish, and sharp cutting edges should be provided on the base. It is very important that no voids form between the sample and cylinder; to facilitate detection of such voids, a volumetric cylinder of transparent Lucite with detachable steel cutting edges may be used.

(2) Guide cylinder for guiding cutter into soil (not absolutely necessary).

(3) Trimming tools, such as wire saw, straightedge, or knife.

(4) Oven (see Appendix I, WATER CONTENT - GENERAL).

(5) Specimen container. The container should be of metal that is resistant to corrosion. Seamless aluminum pans with lids are satisfactory.

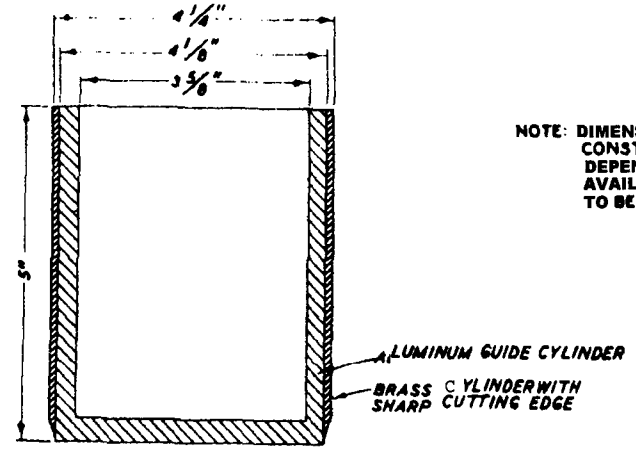
(6) Balance, sensitive to 0.1 g.

(7) Glass plate, large enough to cover top of specimen.

c. Procedure. The procedure shall consist of the following steps:

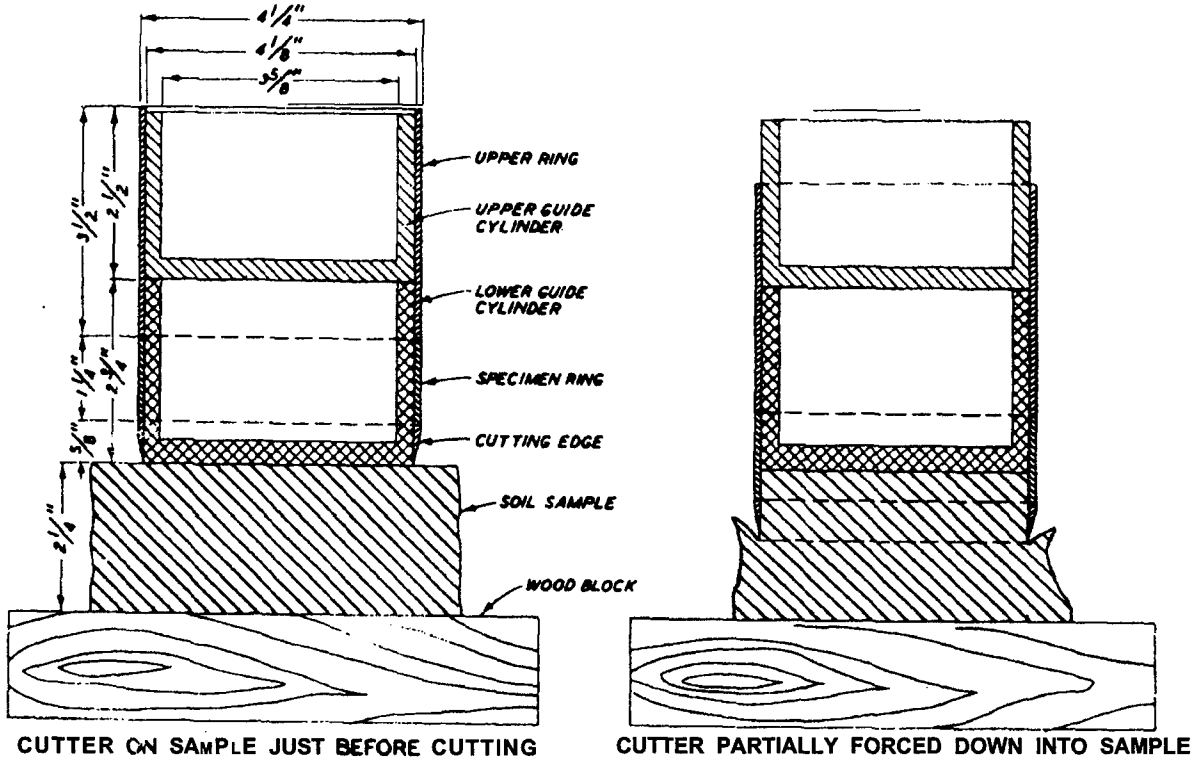
(1) Record on a data sheet (Plate II-1 is a suggested form) all identifying information for the sample, such as project, boring number, and other pertinent data.

(2) Measure and record the height, H, and inside diameter, D, of the volumetric cylinder. In general, linear measurements shall be



NOTE: DIMENSIONS AND MATERIALS OF CONSTRUCTION MAY VARY DEPENDING ON SIZE OF SAMPLES AVAILABLE AND TYPES OF SOILS TO BE TESTED

(a) VOLUMETRIC CYLINDER



CUTTER ON SAMPLE JUST BEFORE CUTTING

CUTTER PARTIALLY FORCED DOWN INTO SAMPLE

(b) VOLUMETRIC CYLINDER OF SPLIT-RING TYPE, SHOWING METHOD OF USE

Figure 2. Examples of volumetric cylinders

made with an accuracy which will result in a volumetric error, dV/V, of less than 1 percent. The volumetric error is represented by the expression:

$$\frac{dV}{V}, \text{ percent} = 200\,\frac{dD}{D} + 100\,\frac{dH}{H}$$

where dH = accuracy of height measurement

dD = accuracy of diameter measure

(3) Center the volumetric cylinder on top of the sample. The sample may be roughly trimmed to a size somewhat larger than the cylinder (see Fig. 2b) or the entire available sample may be used (see Fig. 3). Push the cylinder vertically into the sample not more than 1/4 in. and carefully trim the soil from the edge of the cylinder (see Fig. 3). Repeat the operation until the specimen protrudes above the top, of the calibrated cylinder. Care should be taken that no voids are formed between the cylinder and specimen. Using a wire saw for soft specimens and a knife, straightedge, or other convenient tool for harder specimens, trim the top of the specimen flush with the top of the cylinder. Invert the specimen, place it on a glass plate, and trim the bottom of the specimen.



Figure 3. Determining the unit weight of a soil specimen with the volumetric cylinder (split-ring type)

(4) Remove the specimen from the volumetric cylinder using a guide cylinder, if available, and place it in a container. Weigh the specimen and container and record this weight

on the data sheet as the weight of tare plus wet soil. Alternatively, the wet weight of the specimen may be determined by weighing the volumetric cylinder with the specimen therein and then removing the material and placing it in a container for a water content determination.

(5) Place the soil and container in an oven and oven-dry the specimen at 110 C ± 5, allow it to cool, and then weigh. Record this weight as weight of tare plus dry soil.

d. Computations. (1) Quantities obtained in test. The following quantities are obtained in the test.

(a) Weight of tare (specimen container or cylinder) plus wet soil The tare weight is subtracted from this value to obtain the weight of wet soil, W.

(b) Weight of tare (specimen container plus dry soil. The tare weight is subtracted from this value to obtain the weight of dry soil, W_s, or if the alternate procedure is used, dry weight of specimen is computed by the following formula:

$$\text{Dry weight of specimen} = \frac{\text{wet weight of specimen}}{1 + \left(\frac{\text{water content of specimen}}{100}\right)}$$

$$W_s = \frac{W}{1 + 0.01w}$$

(c) The inside volume of the volumetric cylinder. Volume, V, of the wet soil specimen is equal to this volume. The volume, V, may also be computed from linear measurements of a specimen in the form of a cylinder or parallelepiped.

(2) Unit weights. The wet unit weight, γ_m, and the dry unit weight, γ_d, expressed in terms of pounds per cubic foot, are computed by the following formulas:

$$\text{Wet unit weight} = \frac{\text{weight in g of wet specimen}}{\text{volume in cc of wet specimen}} \times 62.4$$

$$\gamma_m = \frac{W}{V} \times 62.4$$

$$\text{Dry unit weight} = \frac{\text{weight in g of dry specimen}}{\text{volume in cc of wet specimen}} \times 62.4$$

$$\gamma_d = \frac{W_s}{V} \times 62.4$$

(3) Void ratio. The void ratio, e , is computed by the following following formula:

$$\text{Void ratio} = \frac{\text{volume in cc of wet specimen} - \text{volume in cc of solids}}{\text{volume in cc of solids}}$$

$$e = \frac{V - V_s}{V_s}$$

where V_s = volume of solids = $\frac{W_s}{G_s}$

G_s = specific gravity of solids (see Appendix IV, SPECIFIC GRAVITY)

(4) Porosity. The porosity, n, is computed by the following formula:

$$\text{Porosity, percent} = \frac{\text{volume in cc of wet specimen} - \text{volume in cc of solids}}{\text{volume in cc of wet specimen}} \times 100$$

$$n, \text{ percent} = \frac{V - V_s}{V} \times 100$$

(5) Degree of saturation. The degree of saturation, S, is computed by the following formula:

$$\text{Degree of saturation, percent} = \frac{\text{volume in cc of water}}{\text{volume in cc of wet specimen} - \text{volume in cc of solids}} \times 100$$

$$S, \text{ percent} = \frac{V_w}{V - V_s} \times 100$$

where $V_w = W_w$ = difference between the wet weight of the soil specimen and the oven-dried weight

In the metric system, the volume of water, V_w, is approximately equal numerically to the weight of water, W_w.

4. DISPLACEMENT METHOD. a. Description. The displacement method consists of determining the total volume of a soil by measuring the volume or weight of water displaced by the soil mass. The method is particularly adaptable to irregularly shaped specimens and soils containing gravel, shells, etc.

b. Apparatus. The apparatus should consist of the following:

(1) Balance, sensitive to 0.1 g.

(2) Wire basket of sufficient size to contain the soil specimen.

(3) Can, or container, of sufficient size to submerge the wire basket and specimen.

(4) Oven (see Appendix I, WATER CONTENT - GENERAL).

(5) Specimen container. The container should be of metal that is resistant to corrosion, Seamless aluminum pans with lids are satisfactory.

(6) Paintbrush.

(7) Microcrystalline wax or paraffin.†

(8) Container for melting wax, preferably with a self-contained thermostat.

(9) Thermometer, range 0 to 50 C, graduated in 0.1 deg.

c. Procedure. The procedure shall consist of the following steps:

(1) Record all identifying information for the sample, such as

† Among the many microcrystalline waxes found satisfactory are Product 2300 of the Mobil Oil Company, Microwax 75 of the Gulf Oil Corporation, and Wax 1290 of the Sun Oil Company. Paraffin alone is not as suitable for sealing soil specimens because its brittleness and shrinkage upon cooling will cause cracking, especially in thin sections and at corners; a mixture of 50 percent paraffin and 50 percent petrolatum has properties that approach those of a microcrystalline wax.

project, boring number, or other pertinent data, on the data sheet (Plate II-2).

(2) Determine, if not previously established, the specific gravity of the wax to be used. (About 0.9 g per cc, but should be determined for each batch of wax.)

(3) Cut a specimen from the sample to be tested. (The size of the specimen is not very important provided the capacity of the balance is not exceeded. In general, more accurate results will be obtained with larger specimens.) Trim the specimen to a fairly regular shape, Re-entrant angles should be avoided, and any cavities formed by large particles being pulled out should be patched carefully with material from the trimmings.

(4) Determine and record the wet weight of the soil specimen.

(5) Cover the specimen with a thin coat of melted wax, either with a paintbrush or by dipping the specimen in a container of melted wax. Apply a second coat of wax after the first coat has hardened. The wax should be sufficiently warm to flow when brushed on the soil specimen, yet it should not be so hot that it penetrates the pores of the soil. If hot wax comes in contact with the soil specimen it may cause the moisture to vaporize and form air bubbles under the wax.

(6) Determine and record the weight of the wax-coated specimen in air.

(7) Determine and record the submerged weight of the wax-coated specimen. This is done by placing the specimen in a wire basket hooked onto a balance and immersing the basket and specimen in a can of water as shown in Figure 4. In order to directly measure the submerged weight of the wet soil and wax, the balance must have been previously balanced with the wire basket completely submerged in the can of water. Ensure that the specimen is fully submerged, and that the basket is not touching the sides or bottom of the container. Measure the temperature of the water.

(8) Remove the wax from the specimen. It can be peeled off



Figure 4. Determining the weight of a wax-coated specimen submerged in water

after a break is made in the wax surface. Use the entire sample, or as much as is free of wax inclusions, for a water content determination (see Appendix I, WATER CONTENT - GENERAL).

d. Computations. The following quantities are obtained directly in the test:

(1) Weight of uncoated specimen, W.

(2) Weight of soil plus wax. The weight of uncoated specimen, W, is subtracted from this value to obtain the weight of wax.

(3) Weight of soil plus wax in water.

The following computations shall be made:

(1) Divide the weight of the wax by its specific gravity. This gives the volume of the wax.

(2) Subtract the weight of the wax-coated specimen in water from its weight in air. The difference divided by the density of water at the test temperature (see Table IV- 1, Appendix IV, SPECIFIC GRAVITY) is numerically equal to the volume of the coated specimen in cubic centimeters.

(3) Subtract the volume of wax from the volume of the coated specimen to obtain the total volume of the soil specimen, V.

(4) Compute the water content of the specimen (see Appendix I, WATER CONTENT - GENERA-L). If the entire specimen is used for the water content determination, obtain the dry weight of specimen, W_s, directly. If only a portion of the initial specimen is used for the water

content determination, compute the dry weight of specimen according to the following formula:

$$\text{Dry weight of specimen} = \frac{\text{wet weight of uncoated soil}}{1 + \left(\frac{\text{water content of wet soil}}{100}\right)}$$

$$ws = \frac{W}{1 + \frac{w}{100}}$$

Based on the above information, compute the unit weights, void ratio, porosity, and degree of saturation as specified hereinbefore.

5. POSSIBLE ERRORS. Following are possible errors that would cause inaccurate determinations of the total volume:

a. Volumetric Method.. (1) Imprecise measurement of volumetric cylinder (or of cylindrical specimen trimmed by other methods). Three height measurements and nine diameter measurements should be made to determine the average height and diameter of the cylinder. Precise calipers should be used for these measurements rather than flat scales.

(2) Voids formed on side of specimen by trimming beyond cutting edge.

(3) Material lost while removing specimen from cylinder.

b. Displacement Method. Voids on surface of specimen not filled by wax or air bubbles formed beneath wax.

UNIT WEIGHTS, VOID RATIO, POROSITY, AND DEGREE OF SATURATION
(VOLUMETRIC METHOD)

DATE _____

PROJECT _____

BORING NO. _____

WATER CONTENT						
SAMPLE OR SPECIMEN NO.						
TARE NO.						
WEIGHT IN GRAMS	TARE PLUS WET SOIL					
	TARE PLUS DRY SOIL					
	WATER	W_w				
	TARE					
	DRY SOIL	W_s				
WATER CONTENT		w	%	%	%	%
WEIGHT-VOLUME RELATIONS						
SAMPLE OR SPECIMEN NO.						
CYLINDER NO.						
CENTIMETERS	HEIGHT OF CYLINDER	H				
	INSIDE DIAMETER OF CYLINDER	D				
WEIGHT IN GRAMS	WET SOIL AND TARE					
	TARE					
	WET SOIL	W				
	DRY SOIL†	W_s				
SPECIFITY GRAVITY OF SOIL		G_s				
VOLUME IN CC	WET SOIL (VOLUME OF CYLINDER)	V				
	DRY SOIL = W_s/G_s	V_s				
LB PER CU FT	WET UNIT WT = (W/V) 62.4	γ_m				
	DRY UNIT WT = (W_s/V) 62.4	γ_d				
VOID RATIO = $(V - V_s)/V_s$		e				
POROSITY, % = $\left[(V - V_s)/V\right] \times 100$		n	%	%	%	%
DEGREE OF SATURATION, % = $\left[V_w/(V - V_s)\right] \times 100$		S	%	%	%	%

VOLUME OF CYLINDER, $V = \frac{\pi D^2 H}{4}$

VOLUME OF WATER = $V_w = \frac{W_w}{\text{SPECIFIC GRAVITY OF WATER‡}}$

† IF NOT MEASURED ON ENTIRE SPECIMEN, DRY WEIGHT MAY BE COMPUTED AS FOLLOWS: $W_s = \frac{W}{1 + 0.01\ w}$

‡ SPECIFIC GRAVITY OF WATER IN METRIC SYSTEM = 1 (APPROX)

REMARKS _____

TECHNICIAN _____ COMPUTED BY _____ CHECKED BY _____

ENG FORM NO. 3836
REV JUNE 1970

PLATE II-1

UNIT WEIGHTS, VOID RATIO, POROSITY, AND DEGREE OF SATURATION

(DISPLACEMENT METHOD)

DATE _____

PROJECT _____

BORING NO. _____

WATER CONTENT

SAMPLE OR SPECIMEN NO						
TARE NO						
WEIGHT IN GRAMS	TARE PLUS WET SOIL					
	TARE PLUS DRY SOIL					
	WATER	W_w				
	TARE					
	DRY SOIL	W_s				
WATER CONTENT		w	%	%	%	%

WEIGHT-VOLUME RELATIONS

SAMPLE OR SPECIMEN NO.						
TEST TEMPERATURE OF WATER, T, C					°	
WEIGHT IN GRAMS	SOIL AND WAX IN AIR					
	WET SOIL	W				
	WAX					
	WET SOIL AND WAX IN WATER					
	DRY SOIL†	W_s				
SPECIFIC GRAVITY OF SOIL		G_s				
VOLUME IN CC	WET SOIL AND WAX‡					
	WAX					
	WET SOIL	V				
	DRY SOIL = W_s/G_s	V_s				
LB PER CU FT	WET UNIT WT = (W/V) 62.4	γ_m				
	DRY UNIT WT = (W_s/V) 62.4	γ_d				
VOID RATIO = $(V - V_s)/V_s$		e				
POROSITY, % = $[(V - V_s)/V] \times 100$		n	%	%	%	%
DEGREE OF SATURATION, % = $[V_w/(V - V_s)] \times 100$		s	%	%	%	%

VOLUME OF WAX = $\frac{\text{WEIGHT OF WAX}}{\text{SPECIFIC GRAVITY OF WAX}}$ = _____

† IF NOT MEASURED DIRECTLY, MAY BE COMPUTED AS FOLLOWS: $W_s = \frac{W}{1 + 0.01\,w}$

‡ VOLUME OF WET SOIL AND WAX = $\frac{\left(\text{WEIGHT OF WET SOIL AND WAX IN AIR}\right) - \left(\text{WEIGHT OF WET SOIL AND WAX IN WATER}\right)}{\text{DENSITY OF WATER AT TEST TEMPERATURE}}$

REMARKS _____

TECHNICIAN _____ COMPUTED BY _____ CHECKED BY _____

ENG FORM NO. 3837
REV JUNE 1970

PLATE II-2

APPENDIX III:

LIQUID AND PLASTIC LIMITS

* 1. INTRODUCTION

a. The Atterberg Limits. The Atterberg limits are water contents which define the limits of various stages of consistency for fine-grained soils. The liquid limit (LL) and the plastic limit (PL) define the upper and lower limits, respectively, of the plastic range of a soil; the numerical difference between these two limits expresses the plasticity of a soil and is termed the plasticity index (PI). Detailed procedures for determining the liquid and plastic limits for use in classifying soils and developing correlations with engineering properties of soils are given below, and a simplified method for determining the liquid limit is described in Appendix IIIA, ONE-POINT LIQUID LIMIT TEST. A detailed procedure for determining the shrinkage limit is given in Appendix IIIB, SHRINKAGE LIMIT TEST.

b. Definitions.

(1) Liquid Limit. The liquid limit of a soil is the water content, expressed as a percentage of the weight of oven-dried soil at which two halves of a soil pat separated by a groove of standard dimensions will close at the bottom of the groove along a distance of 1/2 in. under the impact of 25 blows in a standard liquid limit device.

(2) Plastic Limit. The plastic limit of a soil is the water content, expressed as a percentage of the weight of oven dried soil at which the soil just begins to crumble into short pieces when rolled into a thread 1/8 in. in diameter.

2. APPARATUS

a. Liquid Limit Device. A mechanical device consisting of a brass cup suspended from a carriage designed to control its drop onto a hard rubber base. A drawing showing the essential features of the device and the critical dimensions is given in Figure 1. The design of the device may vary provided that the essential functions are preserved. The device may be operated either by a hand crank or by an electric motor.

(1) Base. The base shall be hard rubber† having a D Durometer hardness of 80 to 90, and a resilience such that an 8-mm (5/16-in.) diameter polished steel ball, when dropped from a height of 25 cm (9.84 in.) will have an average rebound of at least 80% but no more than 90%. The tests shall be conducted on the finished base with feet attached.

(2) Feet. The base shall be supported by rubber feet designed to provide isolation of the base from the work surface and having an A Durometer hardness no greater than 60 as measured on the finished feet attached to the base.

(3) Cup. The cup shall be brass and have a weight, including cup hanger, of 185 to 215 g.

(4) Cam. The cam shall raise the cup smoothly and continuously to its maximum height, over a distance of at least 180° of cam rotation. The preferred cam motion is a uniformly *

† Micarta No. 221A has been used in the past. It is satisfactory as long as it meets the resilience requirement set forth for hard rubber.

*

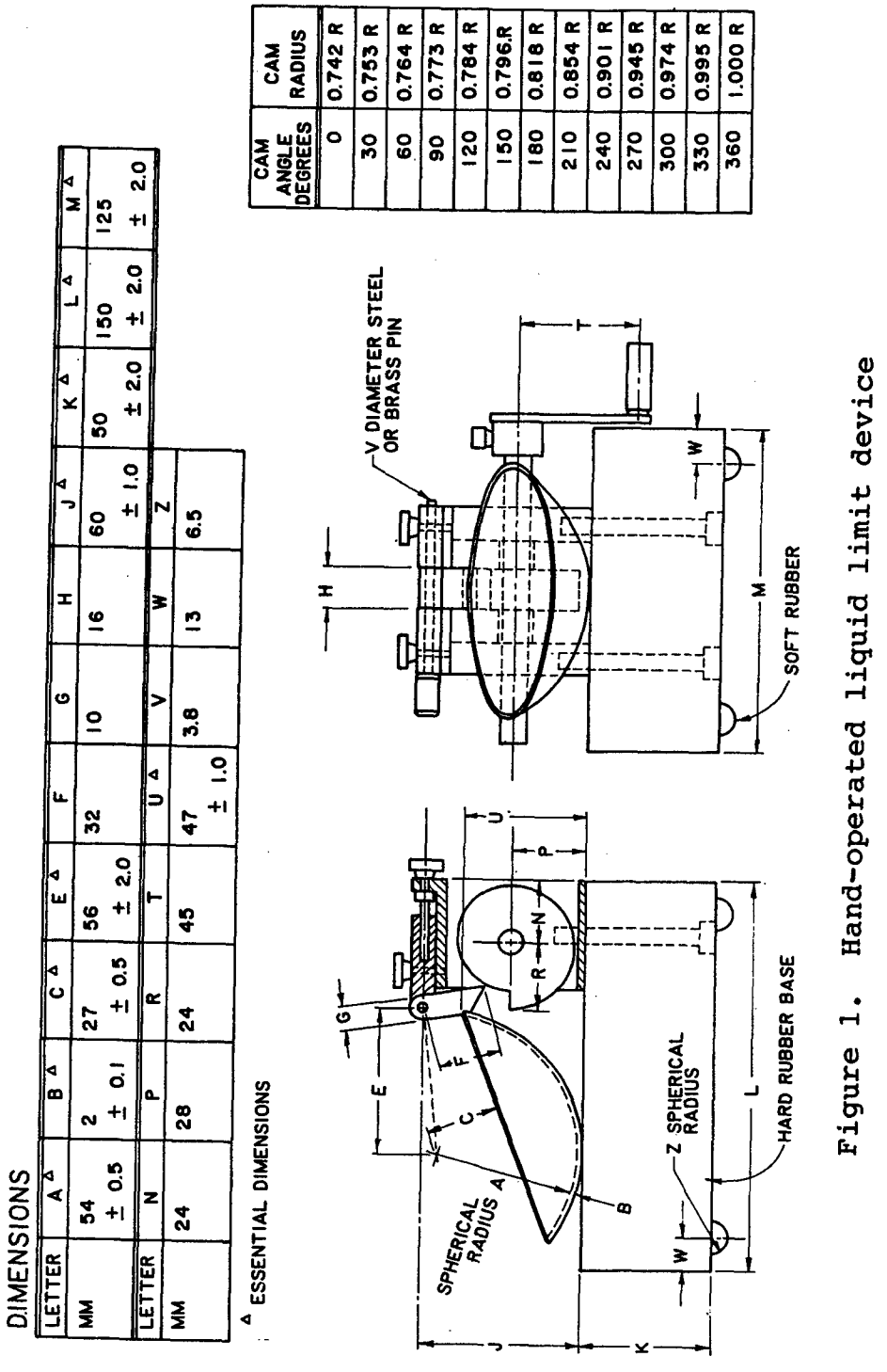


Figure 1. Hand-operated liquid limit device

*

* accelerated lift curve.* The design of the cam and follower combination shall be such that there is no upward or downward velocity of the cup when the cam follower leaves the cam.

(5) Carriage. The cup carriage shall be constructed in a way that allows convenient but secure adjustment of the height of drop of the cup to 10 mm (0.394 in.). The cup hanger shall be attached to the carriage by means of a pin which allows removal of the cup and cup hanger for cleaning and inspection.

(6) Optional Motor Drive. As an alternative to the hand crank shown in Figure 1, the device may be equipped with a motor to turn the cam. Such a motor must turn the cam at 2 ± 0.1 revolutions per second, and must be isolated from the rest of the device by rubber mounts or in some other way that prevents vibration from the motor being transmitted to the rest of the apparatus. It must be equipped with an ON-OFF switch and means of conveniently positioning the cam for height of drop adjustments. The results obtained using a motor-driven device must not differ from those obtained using a manually operated device.

b. Grooving Tool. A grooving tool having dimensions as shown in Figure 2. The tool shall be made of plastic or *

* The cam and follower design in Figure1 is for uniformly accelerated (parabolic) motion. after contact and assures that the cup has no velocity at drop off. Other cam designs also provide this feature and may be used. However, if the cam-follower lift pattern is not known, zero velocity at drop off can be assured by carefully filing or machining the cam and follower so that the cup height remains constant over the last 20 to 45° of cam rotation.

DIMENSIONS

LETTER	A△	B△	C△	D△	E△	F△
MM	2 ± 0.1	11 ± 0.2	40 ± 0.5	8 ± 0.1	50 ± 0.5	2 ± 0.1
LETTER	G	H	J	K△	L△	N
MM	10 MINIMUM	13	60	10 ± 0.05	60 DEG ± 1 DEG	20

△ ESSENTIAL DIMENSIONS

'BACK AT LEAST 15 MM FROM TIP

NOTE: DIMENSION A SHOULD BE 1.9-2.0 AND DIMENSION D SHOULD BE 8.0-8.1 WHEN NEW TO ALLOW FOR ADEQUATE SERVICE LIFE

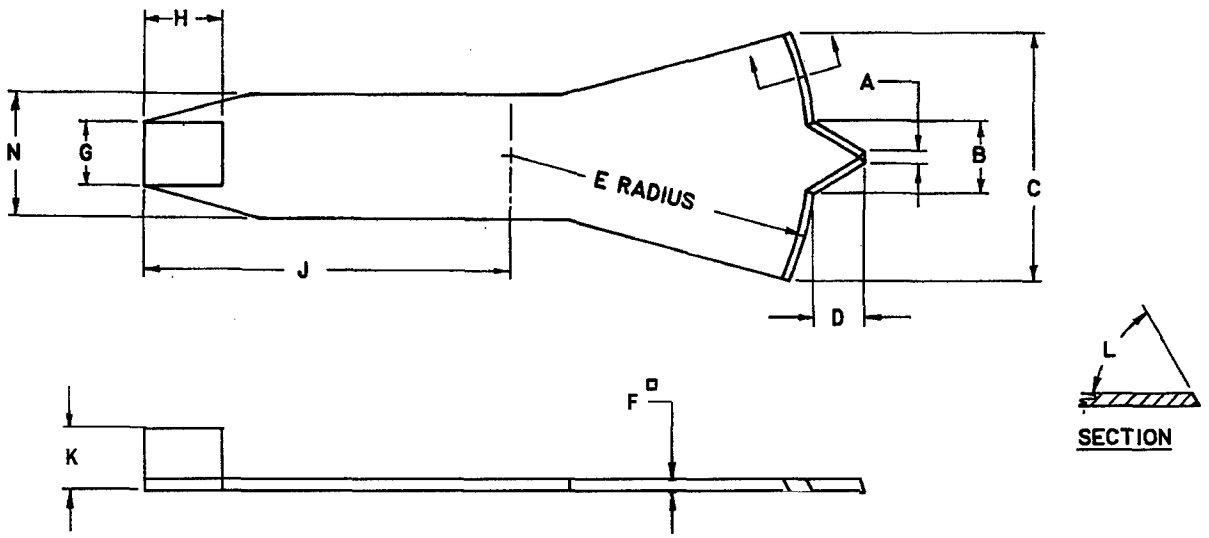
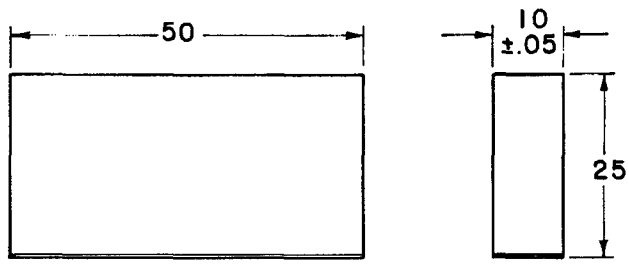


Figure 2. Grooving tool (optional height-of-drop gage attached)

noncorroding metal.* The design of the tool may vary as long as the essential dimensions are maintained. The tool may, but need not, incorporate the gage for adjusting the height of drop of the liquid limit device. *

* **Polycarbonate** plastic grooving tools meeting the dimensional requirements given above are available to US Government agencies through the US Army Engineer Division Laboratory, Southwestern, 4815 Cass Street, Dallas, TX 75235.

* c. **Gage.** A metal gage block for adjusting the height of drop of the cup, having dimensions as shown in Figure 3. The design of the tool may vary provided the gage will rest securely on the base without being susceptible to rocking, and the edge which contacts the cup during adjustment is straight, at least 10 mm (3/8 in.) wide, and without bevel or radius.



DIMENSIONS IN MILLIMETERS

Figure 3. Height of drop gage

d. Containers. Small corrosion-resistant containers with snug-fitting lids for water content specimens. Aluminum or stainless steel cans 2.5 cm (1 in.) high by 5 cm (2 in.) in diameter are appropriate.

e. Balance. A balance readable to at least 0.01 g and having an accuracy of 0.03 g within three standard deviations within the range of use. Within any 15-g range, a difference between readings shall be accurate within 0.01 g.

f. Storage Container. A container in which to store the prepared soil specimen that will not contaminate the specimen in any way, and which prevents moisture loss. A porcelain, glass, or plastic dish about 11.4 cm (4-1/2 in.) in diameter and a plastic bag large enough to enclose the dish and be folded over is adequate. *

* g. Ground Glass Plate. A ground glass plate at least 30 cm (12 in.) square by 1 cm (3/8 in.) thick for mixing soil and rolling plastic limit threads.

h. **Spatula.** A spatula or pill knife having a blade about 2 cm (3/4 in.) wide by about 10 cm (4 in.) long. In addition, a spatula having a blade about 2.5 cm (1 in.) wide and 15 cm (6 in.) long has been found useful for initial mixing of samples.

i. **Sieve.** A 20.3 cm (8 in.) diameter, 425-µm (No. 40) sieve conforming to the requirements of ASTM Specification E11 and having a rim at least 5 cm (2 in.) above the mesh. A 2-mm (No. 10) sieve meeting the same requirements may also be needed.

j. Wash Bottle. A wash bottle or similar container for adding controlled amounts of water to soil and washing fines from coarse particles.

k. Drying Oven. A thermostatically controlled oven, preferably of the forced-draft type, capable of continuously maintaining a temperature of 110 ± 5° C throughout the drying chamber. The oven shall be equipped with a thermometer of suitable range and accuracy for monitoring oven temperature.

l. Washing Pan. A round, flat-bottomed pan at least 7.6 cm (3 in.) deep, slightly larger at the bottom than a 20.3-cm (8-in.) diameter sieve.

m. Rod (Optional). A metal or plastic rod or tube 3.2 mm (1/8 in.) in diameter and about 10 cm (4 in.) long for judging the size of plastic limit threads. *

* n. Mixing Water. A supply of distilled or demineralized water.

o. Blender (Optional). A single speed blender with 1,000 ml container for preparing clay shale materials.

3. CALIBRATION OF APPARATUS

a. Inspection for Wear

(1) Liquid Limit Device. Determine that the liquid limit device is clean and in good working order. The following specific points should be checked:

(2) Wear of Base. The spot on the base where the cup makes contact should be worn no greater than 10 mm (3/8 in.) in diameter. If the wear spot is greater than this, the base can be machined to remove the wear spot provided the resurfacing does not decrease base thickness to less than that specified in 2(a) and the other dimensional relationships are maintained.

(3) Wear of Cup. The cup must be replaced when the grooving tool has work a depression in the cup 0.1 mm (0.004 in.) deep or when the edge of the cup has been reduced to half its original thickness. Verify that the cup is firmly attached to the cup hanger.

(4) Wear of Cup Hanger. Verify that the cup hanger pivot does not bind and is not worn to an extent that allows more than 3-mm (1/8 in.) side-to-side movement of the lowest point on the rim. *

* (5) Wear of Cam. The cam shall not be worn to an extent that the cup drops before the cup hanger (cam follower) loses contact with cam.

(6) Grooving Tools. Inspect grooving tools for wear on a frequent and regular basis. The rapidity of wear depends on the material from which the tool is made and the types of soils being tested. Sandy soils cause rapid wear of grooving tools; therefore, when testing these materials, tools should be inspected more frequently than for other soils. Any tool with a tip width greater than 2.1 mm must not be used. The depth of the tip of the grooving tool must be 7.9 to 8.1 mm. The width of the tip of grooving tools is conveniently checked using a pocket-sized measuring magnifier equipped with a millimeter scale. Magnifiers of this type are available for most laboratory supply companies. The depth of the tip of grooving tools can be checked using the depth measuring feature of vernier calipers.

(7) Blender Blades. Blender blades should be replaced when their overall length becomes 3 mm (1/8 in.) less than their original length.

b. Adjustment of Height of Drop. Adjust the height of drop of the cup so that the point of the cup that comes in contact with the base rises to a height of 10 ± 0.2 mm. A convenient procedure for adjusting the height of drop is as follows: place a piece of masking tape across the outside bottom of the cup parallel with the axis of the cup hanger pivot. The edge of the tape away from the cup hanger should bisect the spot on the cup that contacts the base. For new cups, placing a piece of carbon paper on the base and allowing the cup to drop several times will mark the contact spot. Attach the cup to the device and turn the crank until the cup is raised to its maximum height.*

* Slide the height gage under the cup from the front, and observe whether the gage contacts the cup or the tape (see Figure 4). If the tape and cup are both contacted, the height of drop is approximately correct. If not, adjust the cup until simultaneous contact is made. Check adjustment by turning the crank at 2 revolutions per second while holding the gage in position against the tape and cup. If a ringing or clicking sound is heard without the cup rising from the gage, the adjustment is correct. If no ringing is heard or if the cup rises from the gage1 readjust the height of drop. If the cup rocks on the gage during this checking operation, the cam follower pivot is excessively worn and the worn parts should be replaced. Always remove tape after completion of adjustment operation.

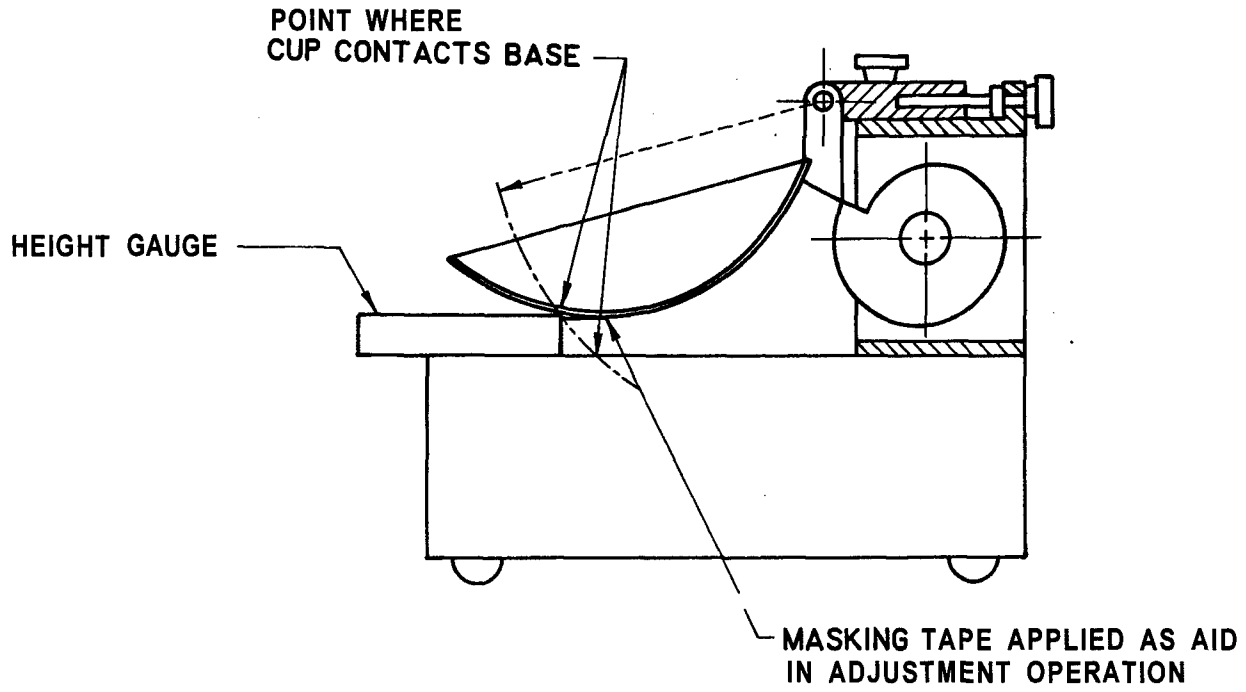


Figure 4. Calibration for height of drop *

* 4. PREPARATION OF MATERIAL

a. Selection of Material. It is essential that the same carefully prepared soil mixture be used for determining both the liquid and plastic limits. Layers of soil of different plasticity should not be mixed. Furthermore, if the natural water content is to be determined, the specimen must be taken from an identical mixture to permit valid correlations. If other test results are to be correlated with the liquid and plastic limits, the material used for the determinations must be the same as that tested. Clay shale materials require special preparation as discussed in Paragraph 9.

b. Effects of Drying. Whenever possible, soils should be at the natural water content when preparation for testing is begun. If drying has occurred before testing; the limit values may change. The plasticity of soils containing organic colloids and certain types of inorganic colloids derived from volcanic rocks is highly sensitive to drying. The effects of drying can be determined by comparing the liquid limit values of specimens in "undried," "airdried," and "ovendried" states.

c. General Preparation of Material

(1) Samples Passing the 425-µm (No. 40) Sieve. When by visual and manual procedures, it is determined that the sample has little or no material retained on the 425-µm (No. 40) sieve, prepare a specimen of 150 to 200 g by mixing thoroughly with distilled or demineralized water on the glass plate using the spatula. If desired, soak soil in a storage dish with small amount of water to soften the soil before the start of mixing. Adjust the water content of the soil to bring it to a consistency that would require 15 to 25 blows of the liquid limit device to *

* close the groove. The time taken to adequately mix a soil will vary greatly depending on the plasticity and initial water content. Initial mixing times of more than 30 min may be needed for stiff, fat clays. If, during mixing, a small percentage of material is encountered that would be retained on a 425-µm (No. 40) sieve, remove these particles by hand, if possible. If it is impractical to remove the coarser material by hand, remove small percentages (less than about 15%) of coarser material by working the specimen through a 425-µm (No. 40) sieve using a piece of rubber sheeting, a rubber stopper, or other convenient device provided the operation does not distort the sieve or degrade material that would be retained if the washing method described in the next paragraph were used. If larger percentages of coarse material are encountered during mixing, or it is considered impractical to remove the coarser material by the methods just described, wash the sample as described in the next paragraph. When the coarse particles found during mixing are concretions, shells, or other fragile particles, do not crush these particles to make them pass a 425-µm (No. 40) sieve, but remove by hand or by washing. Place the mixed soil in the storage dish, cover to prevent loss of moisture, and allow to stand for at least 16 hr (overnight). After the standing period and immediately before starting the test, thoroughly remix the soil.

(2) Samples Containing Material Retained on a 425-µm (No. 40) Sieve

(a) Select a sufficient quantity of soil at natural water content to provide 150 to 200 g of material passing the 425-µm (No. 40) sieve. Place in a pan or dish and add sufficient distilled or demineralized water to cover the soil. Allow to soak until all lumps have softened and the fines no longer adhere to the surfaces of the coarse particles. *

* (b) When the sample contains a large percentage of material retained on the 425-µm (No. 40) sieve, perform the following washing operations in increments, washing no more than 0.5 kg (1 lb) of material at one time. Place the 425-µm (No. 40) sieve in the bottom of the clean pan. Pour the soil water mixture onto the sieve. If gravel or coarse sand particles are present, rinse as many of these as possible with small quantities of water from a wash bottle and discard. Alternatively, pour the soil water mixture over a 2-mm (No. 10) sieve nested atop the 425-µm (No. 40) sieve, rinse the fine material through and remove the 2-mm (No. 10) sieve. After washing and removing as much of the coarser material as possible, add sufficient water to the pan to bring the level to about 13 mm (1/2 in.) above the surface of the 425-µm (No. 40) sieve. Agitate the slurry by stirring with the fingers while raising and lowering the sieve in the pan and swirling the suspension so that fine material is washed from the coarser particles. Disaggregate fine soil lumps that have not slaked by gently rubbing them over the sieve with the fingertips. Complete the washing operation by raising the sieve above the watersurface and rinsing the material retained with a small amount of clean water. Discard material retained on the 425-µm (No. 40) sieve.

(c) Reduce the water content of the material passing the 45-µm (No. 40) sieve until it approaches the liquid limit. Reduction of water content may be accomplished by one or a combination of the following methods: (a) exposing the air currents at ordinary room temperature, (b) exposing to warm air currents from a source such as an electric hair dryer, (c) filtering in a Buckner funnel or using filter candles, (d) decanting clear water from surface of suspension, or (e) draining in a colander or plaster of paris dish lined with high retentivity, *

* high wet-strength filter paper.* If a plaster of paris dish is used, take care that the dish never becomes sufficiently saturated that it fails to actively absorb water into its surface. Thoroughly dry dishes between uses. During evaporation and cooling, stir the sample often enough to prevent overdrying of the fringes and soil pennacles on the surface of the mixture. For soil samples containing soluble salts, use a method of water reduction such as (a) or (b) that will not eliminate the soluble salts from the test specimen.

(d) Thoroughly mix the material passing the 425-um (No. 40) sieve on the glass plate using the spatula. Adjust the water content of the mixture, if necessary, by adding small increments of distilled or demineralized water or by allowing the mixture to dry at room temperature while mixing on the glass plate. The soil should be at a water content that will result in closure of the groove in 15 to 25 blows. Return the mixed soil to the mixing dish, cover to prevent loss of moisture, and allow to stand for at least 16 hr. After the standing period, and immediately before starting the test, remix the soil thoroughly.

5. LIQUID LIMIT

a. Procedure

(1) Place a portion of the prepared soil in the cup of the liquid limit device at the point where the cup rests on the base, squeeze it down, and spread it into the cup to a depth of about 10 mm at its deepest point, tapering to form an *

* S&S 595 filter paper in 32 cm circles has been found satisfactory.

* approximately horizontal surface. Take care to eliminate air bubbles from the soil pat, but form the pat with as few strokes as possible. Heap the unused soil on the glass plate and cover with the inverted storage dish or a wet towel.

(2) Form a groove in the soil pat by drawing the tool, beveled edge, forward through the soil on a line joining the highest point to the lowest point on the rim of the cup. When cutting the groove, hold the grooving tool against the surface of the cup and draw in an arc maintaining the tool perpendicular to the surface of the cup throughout its movement (see Figure 5).

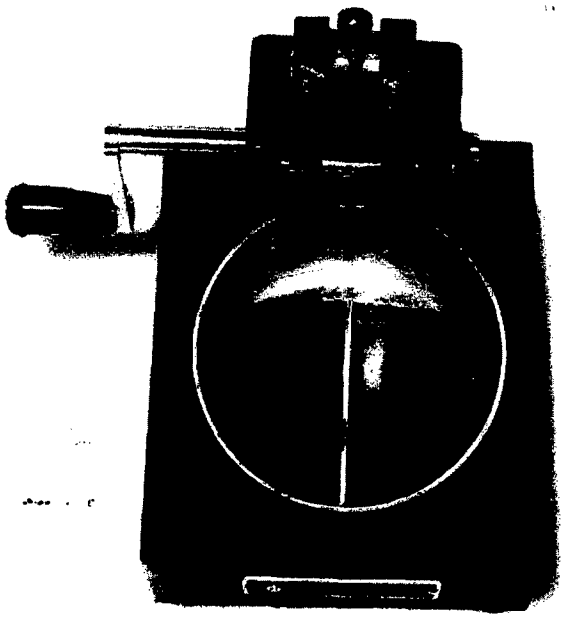


Figure 5. Grooved soil pat in liquid limit device

In soils where a groove cannot be made in one stroke without tearing the soil cut the groove with several strokes of the grooving tool. Alternately, cut the groove to slightly less than required dimensions with a spatula and use the grooving tool to bring the groove to final dimensions. Exercise extreme care to prevent sliding the soil pat relative to the surface or the cup. *

* (3) Verify that no crumbs of soil are present on the base or the underside of the cup. Lift and drop the cup by turning the crank at a rate of 1.9 to 2.1 drops per second until the two halves of the soil pat come in contact at the bottom of the groove along a distance of 13 mm (1/2 in.) (see Figure 6). Use the end of the grooving tool (Figure 2) or a scale to verify that the groove has closed 13 mm (1/2 in.).

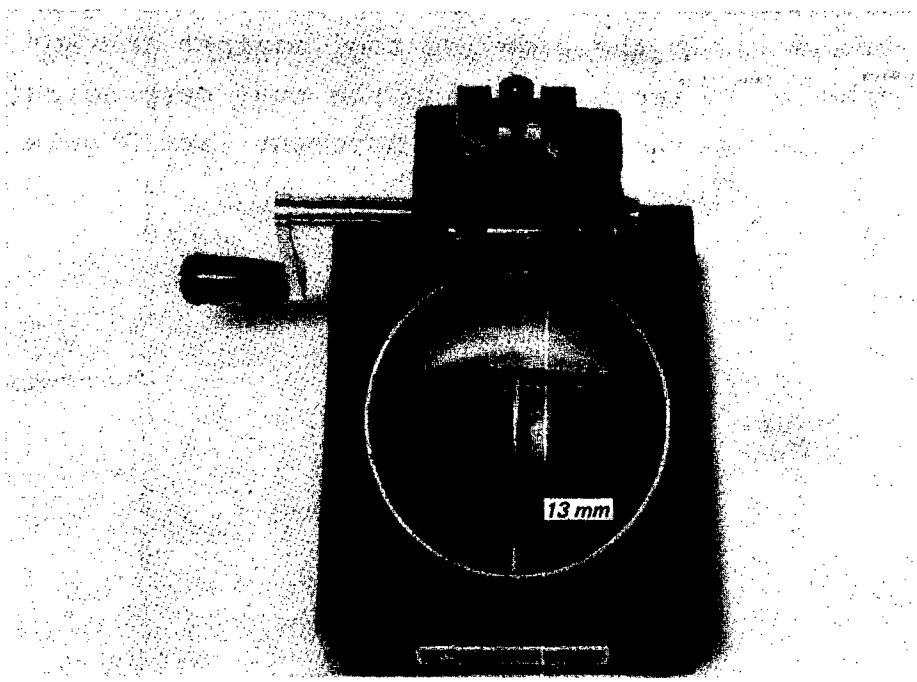


Figure 6. Soil pat after groove has closed

(4) Verify that an air bubble has not caused premature closing of the groove by observing that both sides of the groove have flowed together with approximately the same shape. If a bubble has caused premature closing of the groove, reform the soil in the cup by adding a small, amount of soil to make up for that lost in the grooving operation and repeat 5a(1) through 5a(3). If the soil slides on the surface of the cup, repeat 5a(1) through 5a(3) at a higher water content. If, after several trials at successively higher water contents, the soil pat *

* continues to slide in the cup or if the number of blows required to close the groove is always less than 25, record that the liquid limit could not be determined, and report the soil as non-plastic without performing the plastic limit test.

(5) Record the number of drops (N) required to close the groove. Remove a slice of soil approximately the width of the spatula extending from edge to edge of the soil cake at right angles to the groove and including that portion of the groove in which the soil flowed together, place in a weighed container, and cover.

(6) Return the soil remaining in the cup to the glass plate. Wash and dry the cup and grooving tool and reattach the cup to the carriage in preparation for the next trial.

(7) Remix the entire soil specimen on the glass plate to reduce the water content of the soil and increase the number of blows required to close the groove. Repeat 5a(1) through 5a(6) for at least three additional trials producing successively greater numbers of blows to close the groove. Preferably, two trials should produce closure in 25 blows or less, and two trials should produce closure in 25 blows or more.

(8) Determine the water content (WN) of the soil specimen from each trial in accordance with the procedure in Appendix I, WATER CONTENT - GENERAL. Make all weighings on the same balance. Initial weighings should be performed immediately after completion of the test. If the test is to be interrupted for more than about 15 min, the specimens already obtained should be weighed at the time of the interruption. *

b. Calculations.

(1) Plot the relationship between the water content, W_N, and the corresponding number of drops, N, of the cup on a semilogarithmic graph with the water content as ordinates on the arithmetical scale, and the number of drops as abscissas on the logarithmic scale. See Plate III-1 for an example data form. Draw the best straight line through the four or more plotted points.

(2) Take the water content corresponding to the intersection of the line with the 25-drop abscissa as the liquid limit of the soil. Computational methods may be substituted for the graphical method for fitting a straight line to the data and determining the liquid limit.

6. PLASTIC LIMIT

a. Preparation of Test Specimen. Select a 20-g portion of soil from the material prepared for the liquid limit test, either after the second mixing before the test, or from the soil remaining after completion of the test. Reduce the water content of the soil to a consistency at which it can be rolled without sticking to the hands by spreading and mixing continuously onhe glass plate. The drying process may be accelerated by exposing the soil to the air current from an electric fan or by blotting with paper that does not add any fiber to the soil such as hard surface paper toweling or high wet strength filter paper.

b. Procedure

(1) From the 20-g mass, select a portion of 1.5 to 2.0 g. Form the test specimen into an ellipsoidal mass. Roll *

* this mass between the palm or fingers and the ground-glass plate with just sufficient pressure to roll the mass into a thread of uniform diameter throughout its length. A normal rate of rolling for most soils should be 80 to 90 strokes per minute counting a stroke as one complete motion of the hand forward and back to the starting position. This rate of rolling may have to be decreased for very fragile soils. The thread shall be further deformed on each stroke so that its diameter is continuously reduced; and its length extended until the diameter reaches 3.2 ± 0.5 mm (0.125 ± 0.020 in.), taking no more than 2 minutes to complete the rolling operation. A **3.2-mm (1/8-in.)** diameter rod or tube is useful for frequent comparison with the soil thread to ascertain when the thread has reached the proper diameter especially for inexperienced operators. The amount of hand or finger pressure required will vary greatly according to the soil. Fragile soils of low plasticity are best rolled under the outer edge of the palm or at the base of the thumb. When the diameter of the thread becomes 3.2 mm, break the thread into several pieces. Squeeze the pieces together, knead between the thumb and first finger of each hand, reform into an elliposidal mass, and reroll. Continue this alternate rolling to a thread 3.2 mm in diameter, gathering together, kneading and rerolling, until the thread crumbles under the pressure required for rolling, and the soil can no longer be rolled into a **3.2-mm** diameter thread (see Figure 7). It has no significance if the thread breaks into threads of shorter length. Roll each of these shorter threads to 3.2 mm in diameter. The only requirement for continuing the test is that they are able to be reformed into an ellipsoidal mass and rolled out again. The operator shall at no time attempt to produce failure at exactly 3.2 mm diameter by allowing the thread to reach 3.2 mm, then reducing the rate of rolling or the hand pressure, or both, while continuing the rolling without further deformation until the thread falls apart. It is permissible, *

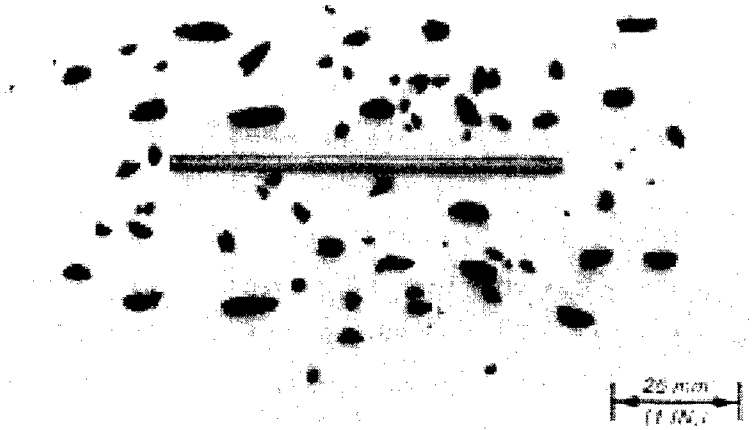


Figure 7. Lean clay soil at the plastic limit

however, to reduce the total amount of deformation for feebly plastic soils by making the initial diameter of the elliposidal mass nearer to the required 3.2 mm final diameter. If crumbling occurs when the thread has a diameter greater than 3.2 mm, this shall be considered a satisfactory end point provided the soil has been previously rolled into a thread 3.2 mm in diameter. Crumbling of the thread will manifest itself differently with the various types of soil. Some soils fall apart in numerous small aggregations of particles, others may form an outside tubular layer that starts splitting at both ends. The splitting progresses toward the middle, and finally,the thread falls apart in many small platy particles. Fat clay soils require much pressure to deform the thread, particularly as they approach the plastic

* limit. With these soils, the thread breaks into a series of barrel-shaped segments about 3.2 to 9.5 mm (1/8 to 3/8 in.) in length.†

(2) Gather the portions of the crumbled thread together and place in a weighed container. Immediately cover the container.

(3) Select another 1.5 to 2.0 g portion of soil from the original 20-g specimen and repeat the operations described in 17.1 and 17.2 until the container has at least 9 g of soil.

(4) Repeat 17.1 through 17.3 to make another container holding at least 9 g of soil. Determine the water content, in percent, of the soil contained in the containers in accordance with the procedure given in Appendix I, WATER CONTENT - GENERAL. Make all weighings on the same balance.

c. Calculations. Compute the average of the two water contents. If the difference between the two water contents is greater than two percentage points, repeat the test. The plastic limit is the average of the two water contents.

7. PLASTICITY INDEX

a. Calculations. Calculate the plasticity index as follows:

$$PI = LL - PL$$ *

† A. Casagrande, R. C. Hirschfield, and S. J. Poulos, Third Progress Report on Investigation of Stress-Deformation and Strength Characteristics of CompactedClays, Soil Mechanics Series No. 70, Harvard University (Cambridge, Mass., November 1963).

* where

LL = the liquid limit

PL = the plastic limit

Both LL and PL are whole numbers. If either the liquid limit or plastic limit could not be determined, or if the plastic limit is equal to or greater than the liquid limit, report the soil as nonplastic, NP.

b. Plasticity chart. Errors in computing the liquid or plastic limits sometimes can be detected by plotting the values of liquid limit versus plasticity index on the plasticity chart? as shown in Figure 8. The upper limit line starts from a liquid limit of 8 at a plasticity index of 0 and rises toward the right with a slope of 9 vertically on 10 horizontally; the equation of the upper limit line, therefore, is PI= 0.9 (LL - 8). A plot of liquid limit versus plasticity index for natural soils has never been known to fall above the upper limit line. Plate III-2 is a suggested form for the graphical correlation of the various Atterberg limits data within a project or testing assignment.

8. REPORT

a. Report the following information:

(1) Sample identifying information, *

† US Army Engineer Waterways Experiment Station, The Unified Soil Classification System, Technical Memorandum No. 3-357, Vol 1 (Vicksburg, Miss., March 1953, revised April 1960). An abridged version of the material in this report is presented in Military Standard MIL-STD-619A, 20 March 1962.

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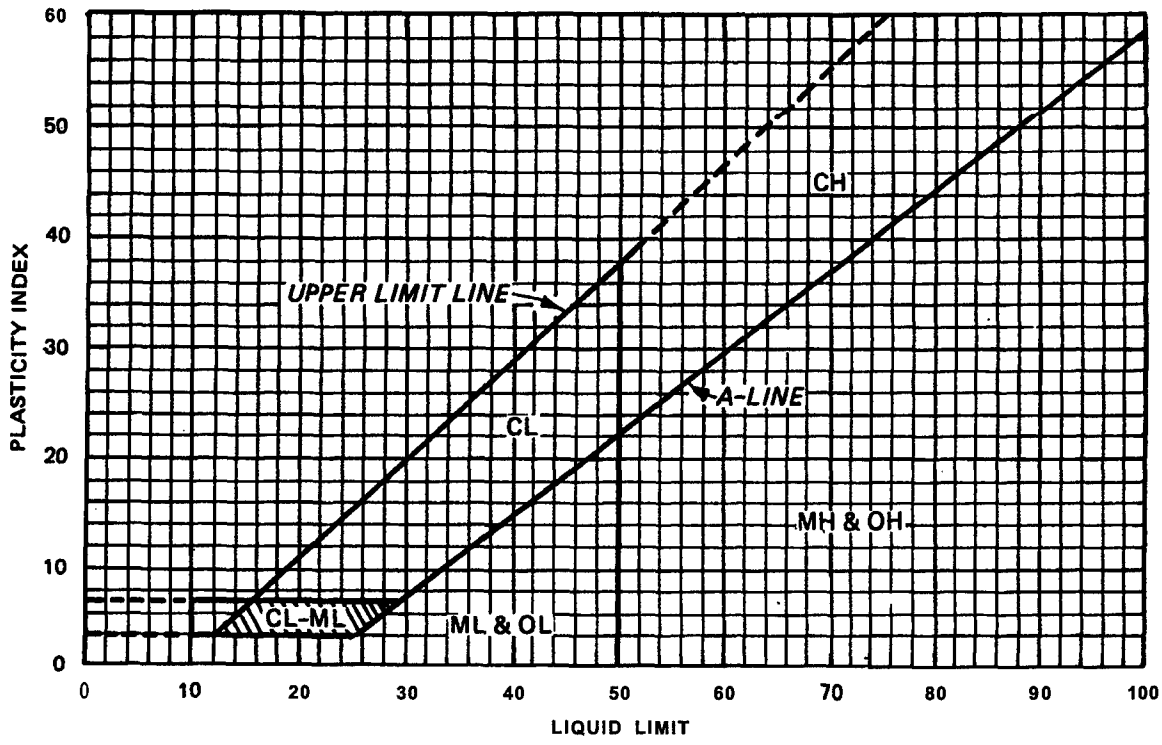


Figure 8. Plasticity chart showing classification group symbols

(2) Any special specimen selection process used such as removal of sand lenses from undisturbed sample,

(3) Liquid limit, plastic limit, and plasticity index to the nearest whole number and omitting the percent designation. If the liquid limit or plastic limit is equal to or greater than the liquid limit, report the soil as nonplastic, NP,

(4) An estimate of the percentage of sample retained on the 425-µm (No. 40) sieve, and

(5) Procedure by which liquid limit was performed, if it differs from the multipoint method. *

* **9.** PREPARING CLAY SHALE MATERIAL FOR TESTING

a. General. Investigations have shown that classification indexes of clay shale materials are affected by air-drying and slaking, by oven-drying and slaking, by the type and duration of mechanical dispersing, and by other variations in procedure. While the methods for preparing clay shale material for testing should cover a sufficient range of disaggregation efforts to assess the strength of interparticle bonds, the number of variables allowed to influence the indexes must be minimized by standardized procedures to prevent the classification of each material becoming a minor research project in itself. Therefore, three standard methods of processing clay shale material will be **used.** These will be referred to as the **blenderized,** undried, and air-dried methods.

The primary method is to test material that has been essentially completely disaggregated by high-speed **blenderizing;** this method will provide a reference value and it should be used for all clay shale samples on which Atterberg limits are to be determined. To provide additional indexes as desired, material that has not been subjected to any drying and material that has been subjected to a single cycle of air-drying and soaking may be tested. These two methods should be used on sufficient representative samples to cover the range of samples identified by the primary method.

b. Standard Methods. When material is to be prepared by all three processing procedures, exercise care that the parent material for the batches is similar. Divide the piece of sample selected by a vertical cut into two parts with one piece about twice as large as the other. Shave the smaller piece into distilled water to produce the undried batch, and use the larger *

* piece to produce the other two batches. Figure 9 shows a flow diagram of the three preparation methods and indicates when separation of batches is required. Material may be taken from each of the three batches and used for Atterberg limits determinations without further processing. Details of each procedure are as follows.

(1) Blenderized (primary method). Shave or shred material at essentially natural water content and dry to a constant weight in an atmosphere with a temperature less than 50° C and a relative humidity less than 30 percent. After a constant weight is attained (and after a drying period of at least 48 hr), soak the material in distilled water for at least 48 hr.

(a) Place about 500 ml of the slurry in the 1,000-ml glass container (available from any laboratory supply company) of a Waring single-speed blender. Make the initial water content of the slurry above 300 percent or more than twice the estimated liquid limit (blenderized), whichever is greater. Typically, the weight of dry soil in the blender at any one time should not exceed 150 g.

(b) Blenderize the slurry without interruption for 10 min and then wash through a 425-µm (No. 40) sieve. Remove excess water using a plaster of Paris dish lined with filter paper. Work material at a water content above the liquid limit in a thin layer on a glass plate with a steel spatula until no further reduction in the size of lumps can be achieved.

(2) Undried. Shave or shred material at essentially natural water content, immediately place in distilled water, and soak for at least 48 hr. After removing excess water by *

*

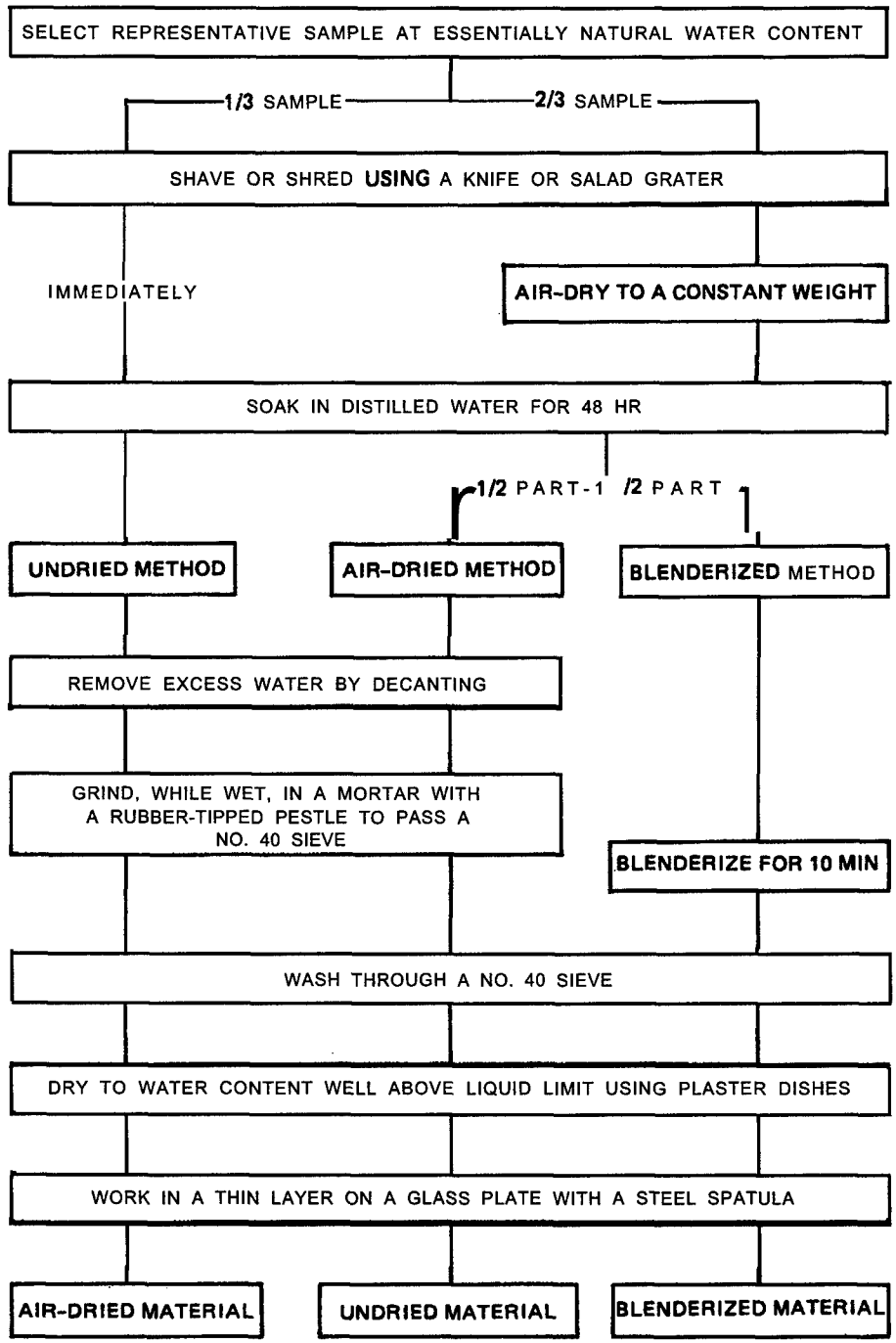


Figure 9. Flow diagram showing the three standard methods of preparing clay shale materials

* decanting, grind the wet material in a mortar with a rubber-tipped pestle and wash through the 425-µm (No. 40) sieve. Remove excess water using a plaster-of-Paris dish lined with filter paper. Work material at a water content above the liquid limit in a thin layer on a glass plate with a steel spatula until no further reduction in the size of lumps can be achieved.

(3) Air-dried. Shave or shred material at essentially natural water content and dry to a constant weight in an atmosphere with a temperature less than 50°C (120°F) and a relative humidity less than 30 percent. After a constant weight is attained (and after a drying period at least 48 hr), soak the material in distilled water for at least 48 hr. After removing excess water by decanting, grind the wet material in a mortar with a rubber-tipped pestle and wash through the 425-µm (No. 40) sieve. Remove excess water using a plaster-of-Paris dish lined with filter paper. Work material at a water content above the liquid limit in a thin layer on a glass plate with a steel spatula until no further reduction in the size of lumps can be achieved.

10. POSSIBLE ERRORS. Following are possible errors that would cause inaccurate determinations of liquid and plastic limits:

a. General.

(1) Specimen not representative. As described in paragraph **4a.**, the liquid and plastic limits must be determined using the same mixture of soil as that used for determinations of natural water content or for other tests. Care should be taken when using the trimmings from preparation of other test specimens that material is as close as possible. *

* (2) Specimen improperly prepared. The specimens must be thoroughly mixed and be permitted to cure for a sufficient period before testing. Erroneous results may be caused by the loss of colloidal material when removing particles coarser than the No. 40 sieve or by testing air-dried or oven-dried soils.

(3) Inaccurate determination of water contents. The possible errors described in paragraph 6 of Appendix I, WATER CONTENT - GENERAL, would greatly affect the computed liquid and plastic limits because of the small quantities of material available for the water content determinations.

(4) Computational mistakes.

b. Liquid Limit Test

(1) Improperly constructed or adjusted liquid limit device.

(2) Worn parts of liquid limit device, especially at point of contact between the cup and the base or worn tip of grooving tool.

(3) Soil at point of contact between the cup and the base. Removal of the cup for shaping and grooving the soil pat will also ensure that the bottom of the cup and the top of the base are clean. Any soil that has dropped onto the base can be removed with one stroke of the back of the hand just before replacing the cup.

(4) Loss of moisture during test. Erratic and erroneous results may be causing by drying of some soil mixtures unless the test is performed in a humid room. *

* c. Plastic Limit Test

(1) Incorrect final thread diameter. A length of 1/8-in.-diameter metal rod close at hand will help in estimating this diameter accurately.

(2) Stopping the rolling process too soon. If there is any doubt as to whether the thread has crumbled sufficiently, it is better to roll the thread once more than to stop the process too soon.

*

LIQUID AND PLASTIC LIMIT TESTS

For use of this form, see EM 1110-2-1906.

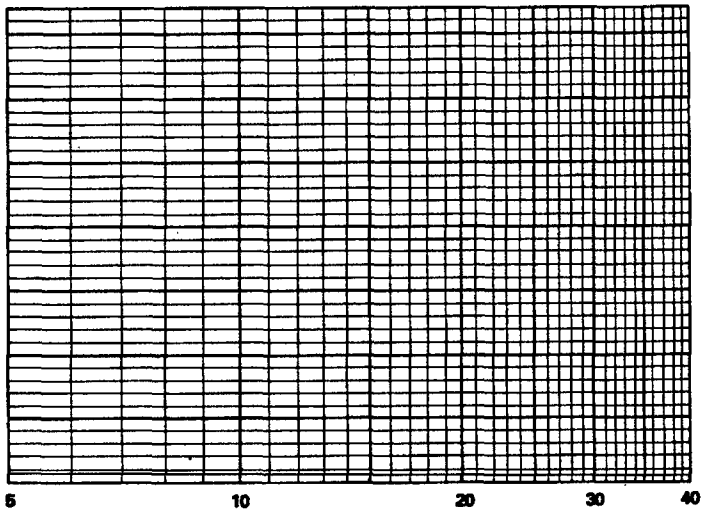
DATE _____

PROJECT _____

BORING NO. _____ SAMPLE NO. _____

LIQUID LIMIT

	RUN NO.	1	2	3	4	5	6
	TARE NO.						
WEIGHT IN GRAMS	TARE PLUS WET SOIL						
	TARE PLUS DRY SOIL						
	WATER W_w						
	TARE						
	DRY SOIL W_s						
	WATER CONTENT, % w						
	NUMBER OF BLOWS						



LL _____

PL _____

PI _____

Symbol from plasticity chart _____

PLASTIC LIMIT

	RUN NO.	1	2	3	4	5	NATURAL WATER CONTENT
	TARE NO.						
WEIGHT IN GRAMS	TARE PLUS WET SOIL						
	TARE PLUS DRY SOIL						
	WATER W_w						
	TARE						
	DRY SOIL W_s						
	WATER CONTENT, % w						
	PLASTIC LIMIT						

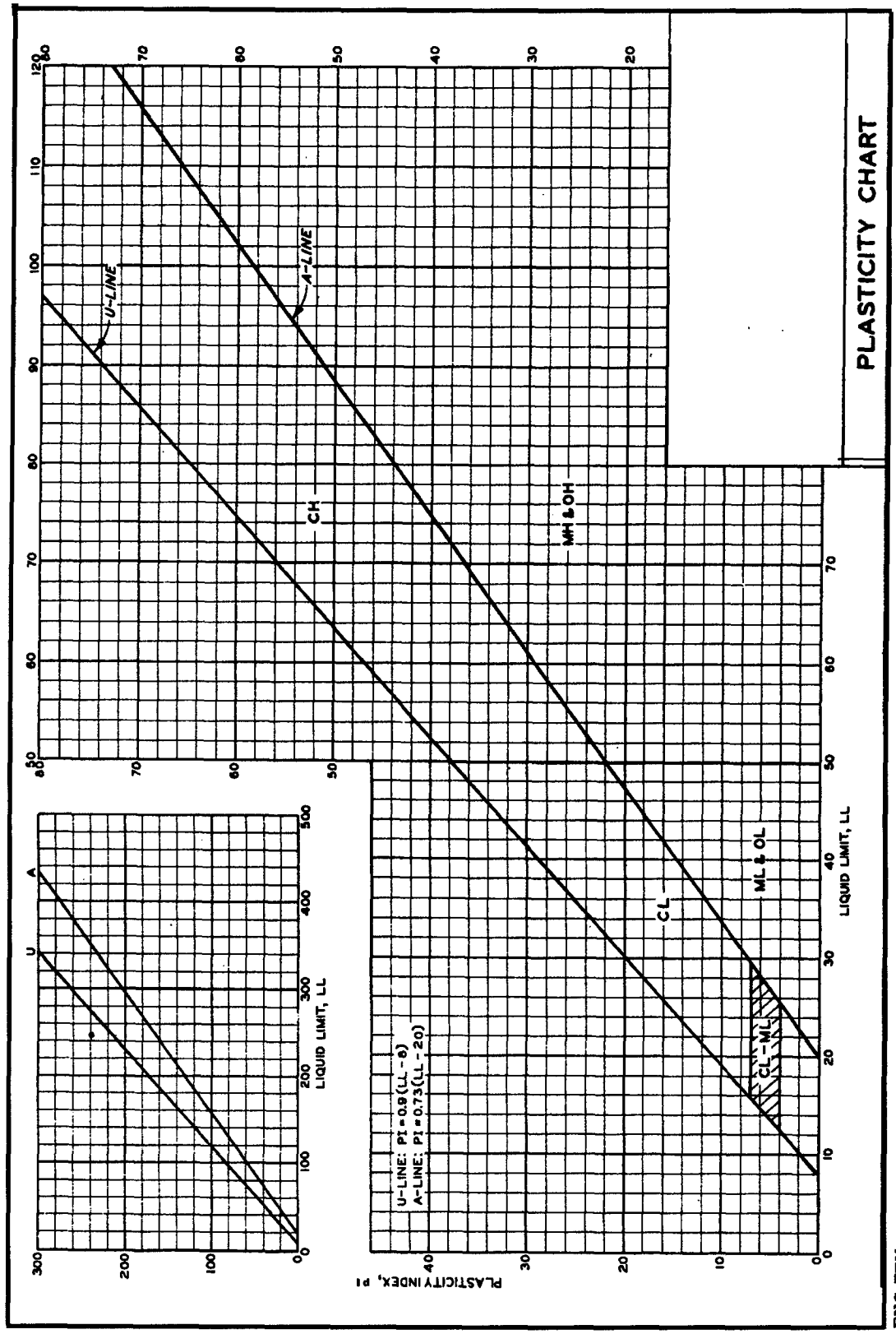
REMARKS _____

TECHNICIAN _____ COMPUTED BY _____ CHECKED BY _____

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*

*


PLASTICITY CHART
U-LINE
A-LINE
CH
MH & OH
CL
ML & OL
CL-ML
U-LINE: PI = 0.9(LL - 8)
A-LINE: PI = 0.73(LL - 20)
PLASTICITY INDEX, PI
LIQUID LIMIT, LL
TRANSLUCENT
ENG FRM 4334 (EM 1110-2-1906)
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*

APPENDIX IIIA:
ONE-POINT LIQUID LIMIT TEST

* 1. INTRODUCTION: The one-point liquid limit test is based on the experience that the slope of the liquid limit flow line for soils within a given geologic environment is essentially a constant on a logarithmic plot. Thus, the liquid limit can be determined from one test point provided the constant defining the slope has been established from correlations on the soil in question. The one-point liquid limit test shall be used only in those areas where the soils are geologically similar and adequate correlations defining the slope of the liquid limit flow line have been made.

This test is best performed by technicians who have experience performing the four-point liquid limit described in Appendix III and who can judge closely the consistency required to cause closure at 20-30 blows. It is generally simpler and faster for inexperienced technicians to perform the four-point limit until experience has been gained.

2. APPARATUS AND PREPARATION OF SAMPLE. The apparatus required is the same as that listed in paragraph 2 of Appendix III, LIQUID AND PLASTIC LIMITS. The sample is prepared in the same manner as that described in paragraph 4 for the standard liquid limit test except that the soil to be tested is prepared initially to a consistency that will require between 20 and 30 blows to cause closure.

3. PROCEDURE

a. Proceed as described in Appendix III, LIQUID AND PLASTIC LIMITS, paragraph 5a(1) through 5a(5) except that the *

* number of blows required to close the groove shall be 20 to 30. If less than 20 or more than 30 blows are required, adjust the water content of the soil and repeat the procedure.*

b. Immediately after removing a water content specimen as described in paragraph 5a(5) of Appendix III, reform the soil in the cup adding a small amount of soil to make up for that lost in the grooving and water content sampling operations. Repeat 5a(1) through 5a(5) of the above appendix, and, if the second closing of the groove requires the same number of drops or no more than two drops difference, secure another water content specimen. Otherwise, remix the entire specimen and repeat.

c. Determine water contents of specimens as described in paragraph 5a(8) of the above appendix.

4. CALCULATIONS

a. (Determine the liquid limit for each water content specimen using the following equation:

$$LL = W_N \left(\frac{N}{25}\right)^{\tan \beta}$$

where

W_N = water content

N = the number of blows causing closure of the groove at water content

$\tan \beta$ = slope of the flow line

---

* Excess drying or inadequate mixing will cause the number of blows to vary.

* For those soils having an average slope of the liquid limit flow line of tan $\beta = 0.121$,† the equation $LL = K(W_N)$ may be used where K = a factor given in Table 1. The liquid limit is the average of the two trial liquid limit values.

b. If the difference between the two trial liquid limit values is greater than one percentage point, repeat the test.

5. POSSIBLE ERRORS. Since this test is identical to the liquid limit test except for the method of calculating the liquid limit, it would be subject to the same possible errors listed in paragraph 10 of Appendix III, LIQUID AND PLASTIC LIMITS.

TABLE 1. Factors for Obtaining Liquid Limit from Water Content and Number of Drops Causing Closure of Groove

N Number of Drops	K Factor for Liquid Limit
20	0.974
21	0.979
22	0.985
23	0.990
24	0.995
25	1.000
26	1.005
27	1.009
28	1.014
29	1.018
30	1.022

† U.S. Army Engineer Waterways Experiment Station, CE, Comparison of Results of Liquid Limit Tests by Standard and One-Point Methods, Miscellaneous Paper No. 3-488 (Vicksburg, Miss., April 1962).

APPENDIX IIIB:

SHRINKAGE LIMIT TEST

1. DEFINITION. The shrinkage limit of a soil is the water content, expressed as a percentage of the weight of the oven-dried soil, at which further loss in moisture will not cause a decrease in its volume. As part of the shrinkage limit test, the shrinkage ratio, R, and linear shrinkage, L_s, are also usually determined. The shrinkage ratio is defined as the ratio between a given volume change and the corresponding change in water content above the shrinkage limit. The linear shrinkage is defined as the decrease in one dimension of a soil mass, expressed as a percentage of the original dimension, when the water content is reduced from a given value to the shrinkage limit.

2. APPARATUS. The apparatus should consist of the following:

a. Evaporating dish; a porcelain evaporating dish approximately 4-1/2 in. in diameter is recommended,

b. Shrinkage dish; a circular porcelain or monel metal dish 1-3/4 in. in diameter and 1/2 in. in height is recommended.

c. Glass cup, about 2 in. in diameter and about 1 in. in height with the top rim ground smooth and flat.

d. Glass plate, 3 in. by 3 in. by 1/16 in. fitted with three metal prongs for immersing the soil pat in mercury as shown in Figure 1.

e. Mercury, sufficient to fill the glass cup to overflowing.

f. Spatula, having a blade about 4 in. long and about 3/4 in. wide.

g. Steel straightedge.

h. Balances, (1) sensitive to 0.01g, and (2) sensitive to 0.1 g.

i. Oven (see Appendix I, WATER CONTENT - GENERAL).

3. PREPARATXON OF MATERIAL. Approximately 30 g of soil shall be obtained from the thoroughly mixed portion of the material passing the No. 40 sieve. The material to be used in the test should be prepared in

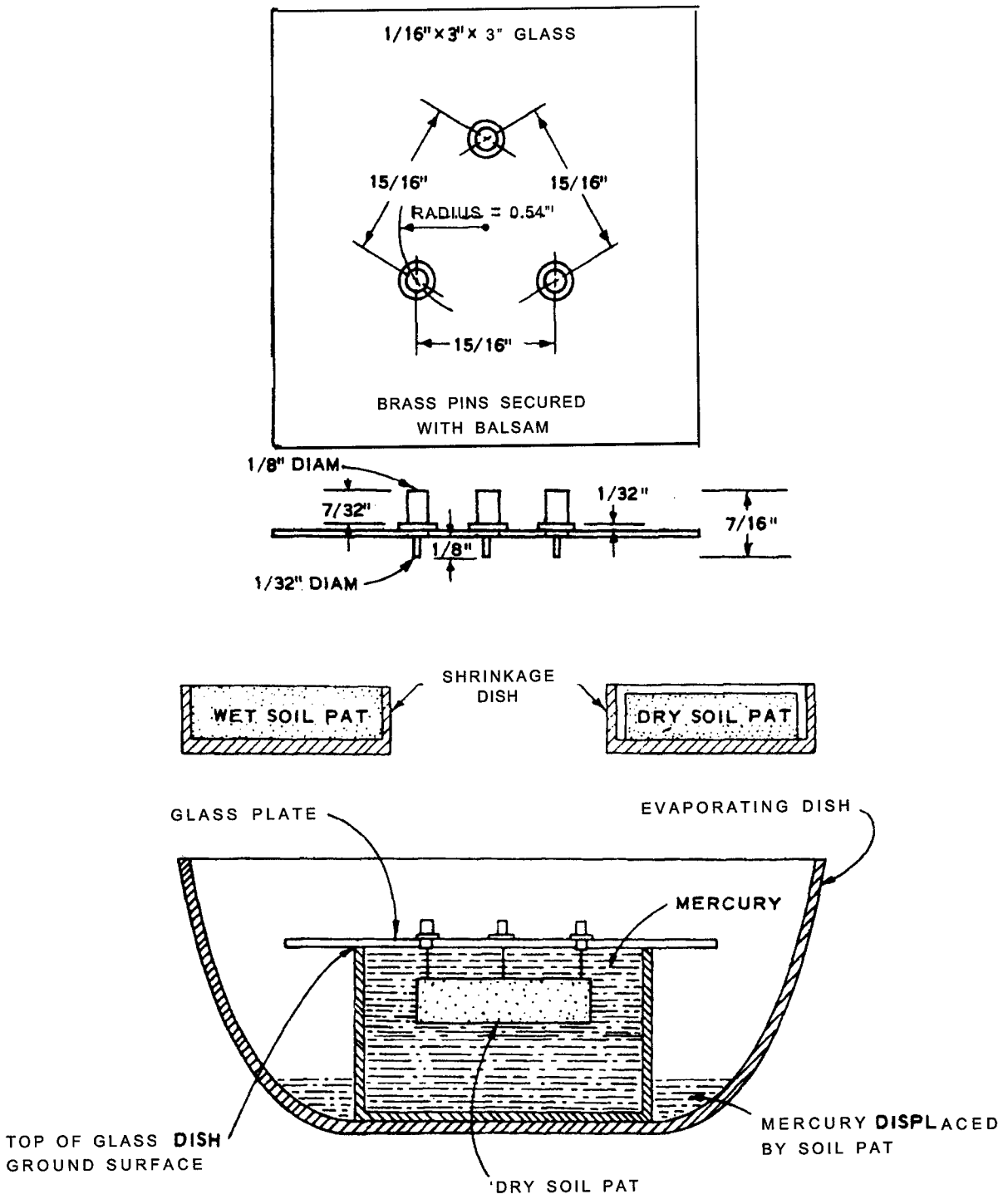


Figure 1. Apparatus for determining the volume of dry soil pat of shrinkage limit test

the same manner as that described for the liquid limit test.

4. PROCEDURE. The procedure shall consist of the following steps:

a. Record all identifying information for the specimen on a data sheet; see Plate IIIB-1 for suggested form.

b. Place the specimen in the evaporating dish and mix it thoroughly with distilled water. The amount of water added shall be sufficient to make the soil wet enough to be readily worked into the shrinkage dish without inclusion of air bubbles. The amount of water required to give friable soils the desired consistency is equal to or slightly greater than the liquid limit; the amount necessary to give plastic soils the desired consistency may exceed the liquid limit as much as 10 percent.

c. Coat the inside surface of the shrinkage dish with a thin layer of petroleum jelly or similar compound to prevent the soil from adhering to the dish. Place an amount of the wetted soil equal to about one-third the volume of the dish in the center of the dish and tap the dish on a firm surface, causing the soil to flow to the outer edges, Continue tapping the dish until all air bubbles are eliminated from the soil. Repeat this step for two more layers. The final layer shall fill the dish completely, with some excess soil allowed to stand above the rim of the dish. Strike off the excess soil with a straightedge and remove all soil adhering to the outside of the dish.

d. Weigh the full dish of soil immediately and record the weight on the data sheet as the weight of dish and wet soil. Allow the soil pat to air-dry until a definite color change takes place and then oven-dry it to a constant weight, Record the oven-dried weight as the weight of the dish and dry soil. Determine and record the weight of the empty dish.

e. Determine the volume of the shrinkage dish by filling the dish to overflowing with mercury,† removing the excess by pressing a glass

† Caution should be exercised in handling mercury. Mercury may have toxic effects, particularly if spilled on the floor in areas without good ventilation. See paragraph 7, HANDLING OF MERCURY.

plate firmly over the top of the dish, and weighing the amount of mercury required to completely fill the dish. The weight of the mercury divided by its density (13.53 g per cc) equals the volume of the inside of the shrinkage dish. Record the volume of the shrinkage dish, which is equal to the volume of the wet soil pat.

f. Place the glass cup in the evaporating dish and fill it with mercury to overflowing. Remove the excess mercury by placing the glass plate with the three metal prongs firmly over the cup; take care not to trap air under the plate. Empty the excess mercury from the evaporating dish and remove all mercury adhering to both the glass cup and the evaporating dish with a brush.

g. Determine the volume of the soil pat by immersing the pat in the mercury contained in the cup, using the glass plate with the three metal prongs as shown in Figure 1. Take care not to trap air under the soil pat or glass plate. Determine the weight of the displaced mercury and compute its volume, as indicated in step e above, and record it as the volume of the dry soil pat.

h. Record all information pertaining to the soil specimen such as weights, volumes, etc., on the data sheet, Plate IIIB-1.

5. COMPUTATIONS. a. Water Content. The water content, w, of the soil at the time it was placed in the shrinkage dish is determined as follows:

$$w = \frac{W_w}{W_s} \times 100$$

where w_w = weight of water in g, obtained by subtracting the weight of the shrinkage dish plus dry soil from the weight of the dish plus wet soil

W_s = weight of oven-dried soil in g, obtained by subtracting the weight of the shrinkage dish from the weight of the dish plus dry soil

b. Shrinkage Limit. The shrinkage limit, **SL**, is calculated as follows:

$$SL = w - \left(\frac{V - V_s}{W_s} \times 100 \right)$$

where **SL** = shrinkage limit

w = water content of wet soil pat when placed in shrinkage dish, expressed as a percentage of the weight of **oven-**dried soil (see paragraph **a)**

V = volume of wet soil pat, cc

$V_s = \frac{\text{weight of displaced mercury in evaporating dish}}{\text{specific gravity of mercury (13.53 g per cc)}}$ = volume of oven-dried soil pat, cc

W_s = weight of oven-dried soil pat, g

c. Shrinkage Ratio, The shrinkage ratio, R, shall be determined by the following formula:

$$R = \frac{W_s}{V_s}$$

where W_s and V_s are the same as given in paragraph **b.**

d. Linear Shrinkage. The linear shrinkage, L_s, shall be determined by the following formula:

$$L_s, 100 \left(1 - \sqrt[3]{\frac{100}{C + 100}} \right)$$

where c = volumetric change from a given water content, w (usually the liquid limit)

$c = (w - SL) R$

6. POSSIBLE ERRORS. Besides errors in the preparation of soil mixtures given in paragraph 4a of Appendix III, LIQUID AND PLASTIC LIMITS, following are possible errors that would cause inaccurate determinations of shrinkage limit:

a. Inside of shrinkage dish not lubricated. If the soil adheres to the shrinkage dish, the soil pat may crack during drying.

b. Air bubbles included in soil pat.

c. Soil pat dried too rapidly. To prevent the soil pat from cracking, it should be dried slowly, first in the humid room and then in the air of the laboratory, until a definite change in color is noted. Only then should it be placed in the oven.

d. Air bubbles trapped beneath soil pat or glass plate when immersing pat in mercury.

7. HANDLING OF MERCURY.* a. Properties. Mercury is an odorless silver-white liquid at normal temperature and pressure. Mercury has the property of forming amalgams with most metals, with the exception of platinum and iron. It is highly volatile, vaporizing at room temperature to form vapors that are highly toxic. Mercury has a specific gravity of 13.6, a boiling point of 356.9 C, and a freezing point of -38.9 C.

b. Health Hazards. The air concentration of mercury vapor corresponding to the equilibrium vapor pressure at room temperature (20 C) is approximately 20 mg per cu m, or 200 times the safe concentration of 0.1 mg per cu m for continuous exposure (8 hr per day, 5 days per week); safe concentrations for shorter periods have not been proposed. Inhalation of mercury vapor of concentrations greater than 0.1 mg per cu m *over* a long period of time can cause chronic poisoning. The initial symptoms of poisoning may include gingivitis, digestive disturbance, fine tremor of the extremities, irritability, excessive emotional response, and

* "Mercury" Data Sheet 203, National Safety Council, 425 N. Michigan Ave., Chicago? Ill. 60611.

exaggerated salivation. These symptoms may increase in severity and may result in permanent disability. Exposure to concentrations far greater than the maximum allowable concentration for short periods of time can cause acute illness. The exact nature, concentration, and duration of exposure determine the type and severity of symptoms, Although inhalation of mercury vapor is by far the greatest avenue of entry to the body, ingestion and absorption through the skin are also possible

c. Use. Whenever possible, use of mercury for teat purposes should be avoided. Personnel working in a n area where mercury is used should be made aware of its hazards.

d. Detection. Direct reading units for determining the concentration of mercury vapor are available from the following sources:

Mine Safety Appliances Company
211 N. Braddock Ave.
Pittsburgh, Pennsylvania 15208

Union Industrial Equipment Corporation (UNICO)
150 Cove St.
Fall River, Massachusetts 02720

Acton Associates
1180 Raymond Blvd.
Newark, New Jersey 07102

Beckman Instruments, Inc.
2500 Harbor Blvd.
Fullerton, California 92634

e. Handling. The precautions listed below should be observed when handling mercury.

(1) Mercury shouldnot be heated without elaborate control because of the rapid increase of vapor pressure with increase in temperature.

(2) Eating and smoking should not be permitted in areas where mercury is handled. Hands should be thoroughly washed after handling mercury.

(3) If clothing becomes contaminated, a change of clothes should be available.

(4) Respiratory protection should be available where there is a possibility of contamination.

(5) All laboratories handling mercury should have a precise plan to be followed in decontamination after a mercury spill. Some general proposals are:

(a) Maximum general exhaust ventilation and local exhaust should be effected. Windows should be opened.

(b) A vacuum flask or a vacuum cleaner designed for removal of mercury should be put into service immediately to recover the mercury.

(c) The area should be decontaminated by treating with flowers of sulfur or other decontaminant.

(d) Effectiveness of decontamination should be verified with a mercury vapor detector.

f. Facilities. In facilities where mercury is handled the measures listed below should be taken.

(1) Floors of areas should be free of cracks and the inter - section of the wall and floor should be fitted with a cove,

(2) Recirculation of air in room should be discouraged because of the possibility of buildup of mercury vapor.

(3) Mercurymanometers should be provided with traps to ensure that there will be no spillage of mercury into a process line or into room.

(4) Precision equipment should be removed from areas where contamination with mercury is possible.

g. Transportation and Storage. Whentransporting and storing mercury the following precautions should be taken.

(1) Mercury containers should be placed in a tray when transported within the laboratory; metal or continuous type (nonwelded) plastic containers are preferable to glass ones.

(2) Mercury containers should be stored in pans that will contain any spillage.

SHRINKAGE LIMIT TEST

Date _____

Project _____

Boring No. _____

Sample or Specimen No.						
Shrinkage Dish No.						
Weight in grams	Dish plus wet soil					
	Dish plus dry soil					
	Water	W_w				
	Shrinkage dish					
	Dry soil	W_s				
	Displaced mercury + evaporating dish					
	Evaporating dish					
	Displaced mercury					
Volume in cc	Shrinkage dish (wet soil pat)	V				
	Volume of dry soil	V_s				
	$V - V_s$					
	$\frac{V - V_s}{W_s} \times 100$					
Water content = $\frac{W_w}{W_s} \times 100$		w	%	%	%	%
Shrinkage limit		SL				
Shrinkage ratio		R				

$$V_s = \frac{\text{weight of displaced mercury}}{\text{specific gravity of mercury (13.53 g/cc)}}$$

SL = Water content of wet soil pat

$$- \left(\frac{\text{volume of wet soil pat - volume of oven-dry soil pat}}{\text{wt of oven-dry soil pat}} \right)$$

$$= w - \left(\frac{V - V_s}{W_s} \times 100 \right)$$

$$R = \frac{\text{wt of oven-dry soil pat}}{\text{volume of oven-dry soil pat}} = \frac{W_s}{V_s}$$

Classification: _____

_____

Remarks _____

_____

Technician _____ Computed by _____ Checked by _____

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PLATE IIIB-1

APPENDIX IV:

SPECIFIC GRAVITY

1. TYPES OF SPECIFIC GRAVITY. The specific gravity of a soil mass for use in soils engineering calculations is usually expressed in three different forms: (a) the specific gravity of solids, G_s, applied to soils finer than the No. 4 sieve; (b) the apparent specific gravity, G_a; and (c) the bulk specific gravity, G_m, both (b) and (c) being applied to soils coarser than the No. 4 sieve. The specific gravity of solids is not applied to coarse particles because they normally contain voids from which air cannot be displaced unless the particles are ground into finer particles to eliminate the voids. Thus, when dealing with coarser particles it is more convenient to work with the apparent specific gravity of the particle mass. The value G_s or G_a is used in all calculations involving fundamental properties of a soil mass. The bulk specific gravity is used in special calculations, such as corrections of density and water content for soils containing gravel sizes. Definitions and detailed procedures for determining the values of specific gravity of solids, apparent specific gravity, and bulk specific gravity are given below.

2. SPECIFIC GRAVITY OF SOLIDS, a. Definition. The specific gravity of solids, G_s, of a soil is the ratio of the weight in air of a given volume of soil particles at a stated temperature to the weight in air of an equal volume of distilled water at a stated temperature.

b. Apparatus. The apparatus should consist of the following:

(1) Volumetric flask, 500-cc capacity

(2) Vacuum pump, with piping and tubing for connections to each flask (as shown in Figure 2, page 4). The connection to each flask should be provided with a trap to catch any water drawn from the flask.

(3) Oven (see Appendix I, WATER CONTENT - GENERAL)

(4) Balance, sensitive to 0.01 g

(5) Thermometer, range 0 to 50 C, graduated in 0.1 degree

(6) Evaporating dish

(7) Water bath

(8) Sieve, U. S. Standard No. 4 conforming to ASTM Designation: E11, Standard Specifications for Sieves for Testing Purposes

c. Calibration of Volumetric Flask. The volumetric flask shall be calibrated for the weight of the flask and water at various temperatures. The flask and water are calibrated by direct weighing at the range of temperatures likely to be encountered in the laboratory. The calibration procedure is as follows:

(1) Fill the flask with deaired-distilled (or deaired-demineralized) water to slightly below the calibration mark and place in a water bath which is at a temperature between 30 and 35 C. Allow the flask to remain in the bath until the water in the flask reaches the temperature of the water bath. This may take several hours. Remove the flask from the water bath, and adjust the water level in the flask so that the bottom of the meniscus is even with the calibration mark on the neck of the flask. Thoroughly dry the outside of the flask and remove any water adhering to the inside of the neck above the graduation; then weigh the flask and water to the nearest 0.01 g. Immediately after weighing, shake the flask gently and determine the temperature of the water to the nearest 0.1 C by immersing a thermometer to the middepth of the flask.

(2) Repeat the procedure outlined in step (1) at approximately the same temperature. Then make two more determinations, one at room temperature and the other at approximately 5 degrees less than room temperature.

(3) Draw a calibration curve showing the relation between temperature and corresponding weights of the flask plus water. Prepare a calibration curve for each flask used for specific gravity determinations and maintain the curves as a permanent record. A typical calibration

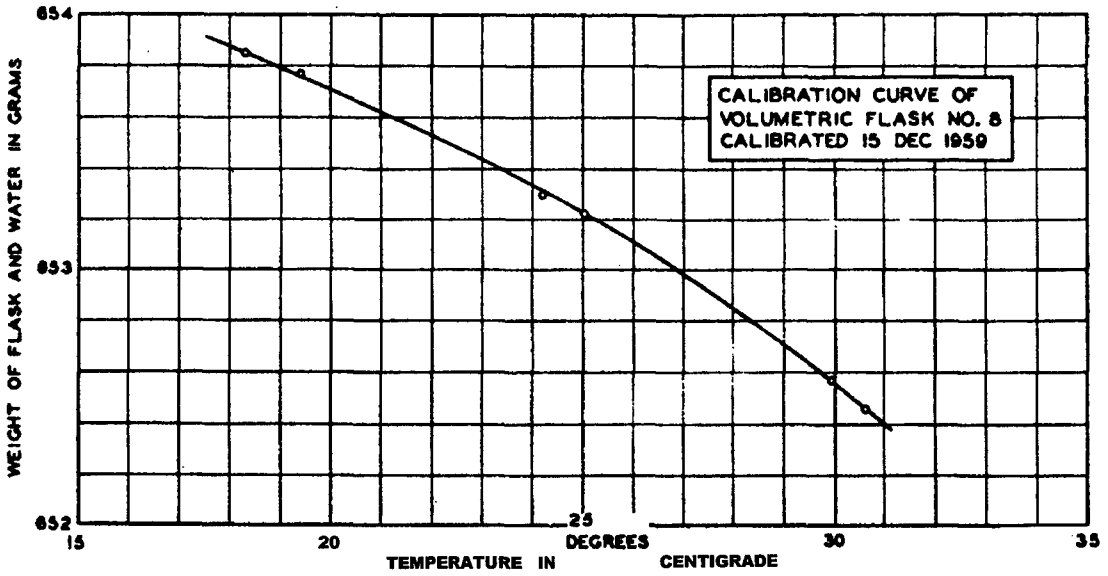


Figure 1. Typical calibration curve of volumetric flask

curve (omitting the fine grid necessary for accurate determinations) is shown in Figure 1.

d. Preparation of Sample. Particular care should be taken to obtain representative samples for determination of specific gravity of solids. The sample of soil may be at its natural water content or oven-dried; however, some soils, particularly those with a high organic content, are sometimes difficult to rewet after having been oven-dried. These soils may be tested without first being oven-dried, in which case the oven-dry weight of sample is determined at the end of the test. When the sample contains particles both larger and smaller than the No. 4 sieve, the sample shall be separated on the No. 4 sieve and a determination made of the specific gravity of the fine fraction and the apparent specific gravity of the coarse fraction. The specific gravity value for the sample shall be the composite specific gravity relation on the basis of the solid volume of the components. When the specific gravity value is to be used in calculations in connection with the hydrometer analysis (see Appendix V, GRAIN-SIZE ANALYSIS), the specific gravity shall be

determined on that portion of the soil used for the hydrometer analysis (usually that which passes the No. 200 sieve). It may be necessary to use other liquids such as kerosene) in lieu of distilled water for testing soils containing soluble salts.

e. Procedure. (1) Soils at natural water content. The procedure for determining the specific gravity of soils at natural water content shall consist of the following steps:

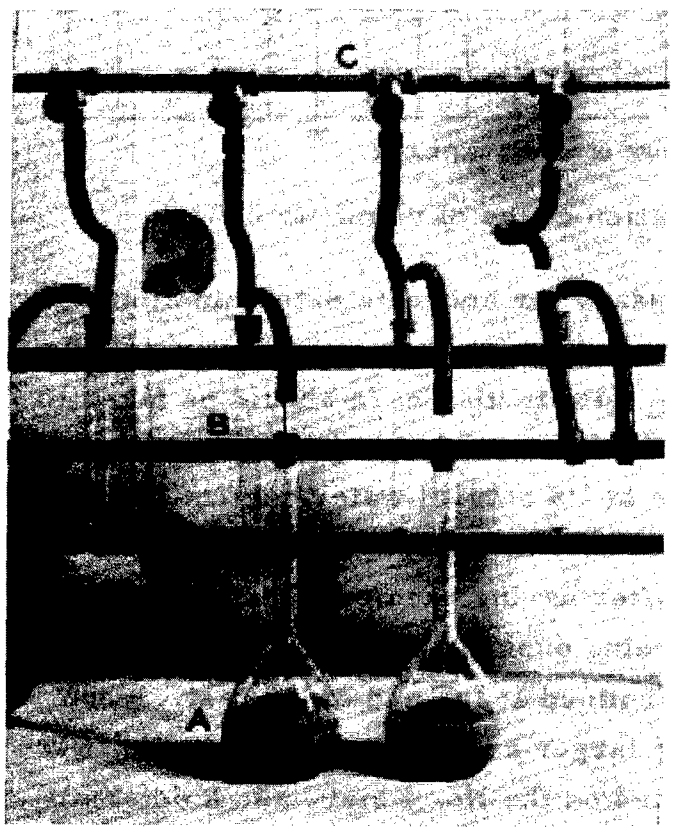


Figure 2. Evacuating air from samples in determination of specific gravity. A, flask; B, splash trap; C, vacuum line

(a) Record all identifying information for the sample such as project, boring number, sample number, and other pertinent data, on a data sheet (see Plate IV-1 for suggested form).

(b) Place a representative sample of soil equivalent to approximately 50 to 80 g oven-dry weight in a dish and, by means of a spatula, mix with sufficient distilled or demineralized water to form a slurry. Place the slurry in a volumetric flask and fill the flask approximately half full with distilled water.

(c) Connect the flask to the vacuum line as shown in Figure 2 and apply a vacuum of approximately 29.0 in. mercury. Agitate the flask gently at intervals during the evacuation process; commercially available mechanical agitators have used for

this purpose. The length of time that vacuum should be applied will depend on the type of soil being tested. Soils of high plasticity and organic † soils usually require 6 to 8 hr; some soils may require less time for removal of air but this should be verified by experimentation. To ensure continuous boiling, the temperature of the flask and contents may be elevated somewhat above room temperature by immersing in a water bath at approximately 35 C. Alternatively, entrapped air may be removed by boiling‡ the suspension gently for at least 10 min while occasionally rolling the flask to assist in the removal of air. The boiling process should be observed closely as loss of material may occur. Allow flask and contents to cool, preferably overnight, before filling and checking.

(d) Fill the flask with deaired distilled water to about 3/4 in. below the 500-cc graduation and apply a vacuum slightly less than that which will cause vigorous boiling (as vigorous boiling may result in a loss of solids). To determine if the suspension is deaired, slowly release the vacuum and observe the lowering of the water surface in the neck of the flask. If the water surface is lowered less than 1/8 in., the suspension can be considered sufficiently deaired.

(e) Fill the flask until the bottom of the meniscus is coincident with the calibration line on the neck of the flask. Thoroughly dry the outside of the flask and *remove* the moisture *on* the inside of the neck by wiping with a paper towel. Weigh the flask and contents to the nearest 0.01 g. Immediately after weighing, stir the suspension to assure uniform temperature, and determine the temperature of the suspension to the nearest 0.1 C by immersing a thermometer to the middepth of the flask.

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† Air removal from organic soils usually cannot be accomplished by the application of vacuum. In this case it will be necessary to boil the suspension contained in the flask for about 30 min, adding distilled or demineralized water carefully from time to time to prevent boiling the sample dry. The flask should at all times be approximately half full.

‡ Use of indirect heat such as a sand bath is recommended.

(f) Carefully transfer the contents of the flask to an evaporating dish. Rinse the flask with distilled water to ensure removal of all of the sample from the flask. Oven-dry the sample to a constant weight at a temperature of. 110 ± 5 C. Allow the soil to cool to room temperature in a desiccator and determine the weight of the soil to the nearest 0.01 g.

(g) Record all weights on the data sheet.

(2) Oven-dried soils. The procedure for determining the specific gravity of solids for oven-dried soils shall consist of the following steps:

(a) Record information identifying the sample on the data sheet (see Plate IV- 1).

(b) Oven-dry the soil to a constant weight at 110 ± 5 C and cool it to room temperature in a desiccator. Select a representative sample of between 50 g (for cohesive soils) and 150 g (for cohesionless soils) and weigh the sample to the nearest 0.01 g. After weighing, transfer the soil to a volumetric flask, taking care not to lose any material during this operation. To avoid possible loss of preweighed soil, the sample may be weighed after transfer to the flask. Fill the flask approximately half full with de-aired distilled water and allow the suspension to stand overnight.

(c) Connect the flask to the vacuum line and apply a vacuum of approximately 29.0 in. mercury for approximately 2 to 4 hr. Entrapped air may also be removed by boiling as previously discussed; however, the process should be observed closely to avoid loss of material during boiling. Allow flask and contents to cool, preferably overnight, before filling and checking.

(d) Perform the remainder of the test as outlined in paragraph 2e(1)(d) and 2e(1)(e).

(e) Record all weights on the data sheet.

f. Computations. The following quantities are obtained by direct weighing :

(1) Weight of flask plus water plus solids at test temperature = W_{bws} in grams.

(2) Weight of tare plus dry soil in grams. The tare weight is subtracted from this value to obtain the weight of dry soil, W_s. The specific gravity of solids is computed to two decimal places by the formula:

$$G_s = \frac{W_s K}{W_s + W_{bw} - W_{bws}}$$

where

K = correction factor based on the density of water at 20 C (see Table IV-1). Unless otherwise required, specific gravity values reported shall be based on water at 20 C.

W_{bw} = weight of flask plus water at test temperature in grams (obtained from calibration curve as shown in Figure 1).

3. APPARENT AND BULK SPECIFIC GRAVITY. a. Definitions. The apparent specific gravity, G_a, of a soil is the ratio of the weight in air of a given volume of the impermeable portion of a permeable material (that is, the solid matter including its impermeable pores or voids) at a stated temperature to the weight in air of an equal volume of distilled water at a stated temperature.

The bulk specific gravity of a soil, G_m, is the ratio of the weight in air of a given volume of a permeable material (including both permeable and impermeable voids normal to the material) at a stated temperature to the weight in air of an equal volume of distilled water at a stated temperature.

b. Apparatus. The apparatus should consist of the following:

(1) Balance, having capacity of 5 kg or more and sensitive to 1.0 g

(2) Wire basket of No. 6 mesh, approximately 8 in. in diameter and 8 in. high

(3) Suitable container for immersing the wire basket in water, and suitable apparatus for suspending the wire basket from

Table IV-1

Relative Density of Water and Correction Factor (K) for Various Temperatures

Temp °C	Relative Density	Correction Factor, K	Temp °C	Relative Density	Correction Factor, K	Temp °C	Relative Density	Correction Factor, K
18.0	0.99862	1.0004	23.0	0.99756	0.9993	28.0	0.99626	0.9980
.1	60	4	.1	54	3	.1	23	0
.2	58	4	.2	51	3	.2	20	0
.3	56	3	.3	49	3	.3	17	0
.4	54	3	.4	46	2	.4	14	79
.5	52	3	.5	44	2	.5	11	9
.6	50	3	.6	42	2	.6	08	9
.7	49	3	.7	39	2	.7	06	8
.8	47	2	.8	37	1	.8	03	8
.9	45	2	.9	34	1	.9	00	8
19.0	0.99843	1.0002	24.0	0.99732	0.9991	29.0	0.99597	0.9977
.1	41	2	.1	29	1	.1	94	7
.2	39	2	.2	27	0	.2	91	7
.3	37	1	.3	24	0	.3	88	6
.4	35	1	.4	22	0	.4	85	6
.5	33	1	.5	20	0	.5	82	6
.6	31	1	.6	17	89	.6	79	6
.7	29	1	.7	14	9	.7	76	5
.8	27	0	.8	12	9	.8	73	5
.9	25	0	.9	09	9	.9	70	5
20.0	0.99823	1.0000	25.0	0.99707	0.9988	30.0	0.99567	0.9974
.1	21	0	.1	04	8	.1	64	4
.2	19	0	.2	02	8	.2	61	4
.3	17	0.9999	.3	699	8	.3	58	3
.4	15	9	.4	97	7	.4	55	3
.5	13	9	.5	94	7	.5	52	3
.6	10	9	.6	91	7	.6	49	3
.7	08	8	.7	89	7	.7	46	2
.8	06	8	.8	87	6	.8	43	2
.9	04	8	.9	84	6	.9	40	2
21.0	0.99802	0.9998	26.0	0.99681	0.9986	31.0	0.99537	0.9971
.1	00	8	.1	78	6	.1	33	1
.2	798	8	.2	76	5	.2	30	1
.3	96	7	.3	73	5	.3	27	0
.4	93	7	.4	70	5	.4	24	0
.5	91	7	.5	68	4	.5	21	0
.6	89	7	.6	65	4	.6	18	69
.7	87	6	.7	63	4	.7	15	9
.8	85	6	.8	60	4	.8	12	9
.9	83	6	.9	57	3	.9	09	9
22.0	0.99780	0.9996	27.0	0.99654	0.9983	32.0	0.99505	0.9968
.1	78	6	.1	51	3	.1	02	8
.2	75	5	.2	48	2	.2	499	8
.3	73	5	.3	46	2	.3	96	7
.4	70	5	.4	43	2	.4	93	7
.5	68	5	.5	40	2	.5	90	7
.6	65	4	.6	37	1	.6	86	6
.7	63	4	.7	34	1	.7	83	6
.8	61	4	.8	32	1	.8	80	6
.9	58	4	.9	29	1	.9	77	5

* Note: Relative density of water based on density of water at 4 C equal to unity. The values given are numerically equal to the absolute density in grams/milliliter (for soil testing purposes, g/ml = g/cc). Data obtained from Smithsonian Tables, compiled by various authors.

Correction factor, K, is found by dividing the relative density of water at the test temperature by the relative density of water at 20 C.

the center of the balance scale pan

(4) Thermometer, range 0 to 50 C, graduated in 0.1 degree

c. Sample. The material to be tested shall be separated on the No. 4 sieve and the material retained on the sieve used for the test. A representative sample of approximately 2 kg is required. Samples may be air -dried; however, oven-drying the sample before the test may affect the results and should be avoided when possible.

d. Procedure. The procedure for determining the apparent and bulk specific gravity of soils retained on the No. 4 sieve shall consist of the following steps:

(1) Record information identifying the specimen on the data sheet (see Plate IV-1).

(2) Wash the specimen thoroughly to remove dust or other coatings from the surfaces of the particles.

(3) Immerse the specimen in water at 15 to 25 C for a period of 24 hr.

(4) Remove the specimen from the water and roll it in a large absorbent cloth until all visible films of water are removed, although the surfaces of the particles may still appear to be damp. Wipe large particles individually. Take care to avoid excess evaporation during the operation of surface drying.

(5) Obtain the weight in grams of the saturated surface-dry specimen. The specimen in this and subsequent weighings should be weighed to the nearest 1.0 g.

(6) Immediately after weighing, place the specimen in the wire basket and determine the weight of the specimen in water. Deter - mine and record the temperature of the water in which the specimen is immersed.

(7) Oven-dry the specimen to a constant weight at 110±5 C. After cooling to room temperature, weigh the specimen.

(8) Record all weights on the data sheet.

e. Computations. The following quantities are obtained by direct weighing:

(1) Weight of tare plus oven-dried soil in grams. The tare weight is subtracted from this value to obtain the weight of dry soil, A, on the data sheet.

(2) Weight of tare plus saturated surface-dry soil in grams. The tare weight is subtracted from this value to obtain the weight of saturated surface-dry soil, B.

(3) Weight of wire basket plus saturated soil in water in grams. The weight of wire basket in water is subtracted from this value to obtain the weight of saturated soil in water, C.

The apparent specific gravity is computed to two decimal places by the formula:

$$G_a = \frac{AK}{A - C}$$

where

K = correction factor based on the density of water at 20 C (see Table IV-i).

The bulk specific gravity is computed to two decimal places by the formula:

$$G_m = \frac{AK}{B - C}$$

When a soil is composed of particles both larger and smaller than the No. 4 sieve, the specific gravity of the soil for use in engineering calculations shall be computed as follows:

$$G = \frac{100}{\frac{\% \text{ passing No. 4 sieve}}{G_s} + \frac{\% \text{ retained on No. 4 sieve}}{G_a}}$$

4. POSSIBLE ERRORS. Following are possible errors that would cause inaccurate determinations of specific gravity:

a. Specific Gravity of Solids. (1) Imprecise weighing of

flask and contents. Since the computation of the specific gravity of solids is based on a difference in weights which is small in comparison with the weights themselves, the same balance should be used for calibrating the volumetric flask and for determining the specific gravity whenever the calibration curve is used.

(2) Temperature of flask and contents not uniform. Both in calibrating the flask and determining the specific gravity, utmost care should be taken to insure that measured temperatures are representative of the flask and contents during the times when the weighings are made.

(3) Flask not clean. The calibration curve will not remain valid if dirt accumulation changes the weight of the flask. Also, if the inside of the neck is not clean, an irregular meniscus may form,

(4) Moisture on outside of flask or inside of neck. When calibrating the flask for a temperature lower than room temperature, there is a tendency for condensation to form on the flask despite careful drying and rapid weighing. Whenever possible, weighing should be done at approximately the same temperature as that of the flask.

(5) Meniscus not coincident with mark on neck of flask. One drop of water too much makes an error of approximately 0.05 g. This error can be minimized by taking the average of several readings at the same temperature. When the suspension is opaque, a strong light behind the neck is helpful in seeing the bottom of the meniscus.

(6) Use of water containing dissolved solids. It is essential that distilled or demineralized water be used exclusively to insure the continued validity of the flask calibration curve.

(7) Incomplete removal of entrapped air from soil suspension. This is the most serious source of error in the specific gravity determination and will tend to lower the computed specific gravity. The suspension must be thoroughly evacuated or boiled and the absence of entrapped air verified as described in paragraph 2e(1)(d). (It should be noted that air dissolved in the water will not affect the results, so it is not

necessary to apply vacuum to the flask when calibrating or after filling the flask to the calibration mark.)

(8) Gain in moisture of oven-dried specimen before weighing. If the specimen is oven-dried before the specific gravity determination, it must be protected against a gain in moisture until it can be weighed and placed in the flask.

(9) Loss of material from oven-dried specimen. If the specimen is oven-dried and weighed before being placed in the flask, any loss of material will lower the computed specific gravity.

b. Apparent and Bulk Specific Gravity. (1) Loss of moisture from saturated surface-dry particles before weighing. Unless the saturated surface-dry material is weighed promptly, evaporation may cause an increase in the computed bulk specific gravity,

(2) Failure to correct for the change in density of water with temperature. This correction is often overlooked when computing either the apparent or bulk specific gravity.

SPECIFIC GRAVITY TESTS

Date _____

Project _____

Boring No. _____

SPECIFIC GRAVITY OF SOLIDS (G_s)

Sample or Specimen No.						
Flask No.						
Temperature of water and soil, T, °C						
Dish No.						
Weight in grams: Dish + dry soil						
Dish						
Dry soil	W_s					
Flask + water at T, °C	W_{bw}					
$W_s + W_{bw}$						
Flask + water + immersed soil	W_{bws}					
Displaced water, $W_s + W_{bw} - W_{bws}$						
Correction factor	K					
$(W_s K) \div (W_s + W_{bw} - W_{bws})$	G_s					

APPARENT (G_a) AND BULK (G_m) SPECIFIC GRAVITY

Sample or Specimen No.						
Temperature of water and soil, T, °C						
Weight in grams: Tare + saturated surface-dry soil						
Tare						
Saturated surface-dry soil	B					
(Wire basket + soil) in water						
Wire basket in water						
Saturated soil in water	C					
Tare + dry soil						
Tare						
Dry soil	A					
Correction factor	K					
$(AK) \div (A - C)$ (Apparent)	G_a					
$(AK) \div (B - C)$ (Bulk)	G_m					

Remarks _____

Technician _____ Computed by _____ Checked by _____

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PLATE IV-i

APPENDIX V:

GRAIN-SIZE ANALYSIS

1. DEFINITION. Grain-size analysis is a process in which the proportion of material of each grain size present in a given soil (grain-size distribution) is determined. The grain- size distribution of coarse -grained soils is determined directly by sieve analysis, while that of fine-grained soils is determined indirectly by hydrometer analysis. The grain-size distribution of mixed soils is determined by combined sieve and hydrometer analyses, Detailed procedures for determining the grain-size distribution of soils by sieve, hydrometer, and combined analyses are given below.

2. SIEVE ANALYSIS. a. Description. A sieve analysis consists of passing a sample through a set of sieves and weighing the amount of material retained on each sieve, Sieves are constructed of wire screens with square openings of standard sizes. The sieve analysis is performed on material retained on a U. S. Standard No. 200 sieve. The sieve analysis, in itself, is applicable to soils containing smali amounts of material passing the No. 200 sieve provided the grain-size distribution of that portion of the sample passing the No. 200 sieve is not of interest.

b. Apparatus. The apparatus should consist of the following:

(1) A series of U. S. standard sieves with openings ranging from 3 in. to 0.074 mm (No. 200), including a cover plate and bottom pan, conforming to ASTM Designation: E ii, Standard Specifications for Sieves for Testing Purposes.* The number and sizes of sieves used for testing a given soil will depend on the range of soil sizes in the material, and the intended use of the gradation curve.

(2) Sieve shaker, a mechanical unit which can produce on duplicate samples the same consistent results as those obtained by the circular and tapping motion used in hand sieving. Typical commercially available mechanical shakers are the Tvler "Ro-Tap" and the Combs and

* See page V-26 for U. S. Standard Sieve Sizes or numbers and sieve openings in inches and millimeters.

Syntron machines; there appears to be no significant differences in the resultsobtained among these machines.†

(3) Balances, sensitive to 0.1 g for samples weighing less than 500 g, and to 1.0 g for samples weighing over 500 g.

(4) Paintbrush, 1 in., or soft wire brush, for cleaning sieves.

(5) Sample splitter or riffle for dividing samples.

(6) Mortar and rubber-covered pestle, for breaking up aggregations of soil particles.

(7) Oven, similar to that described in Appendix I, WATER CONTENT - GENERAL.

c. Preparation of Sample.‡ The material to be treated is first air-dried, after which the aggregations present in the sample are thoroughly broken up with the fingers or with the mortar and pestle. A representative sample is then obtained by dividing, using the sample splitter or riffle. The size of the sample to be used will depend on the maximum particle size in the sample and the requirement that the sample be representative of the material to be tested. The sample should be limited .in weight so that no sieve in the series will be overloaded. Overloading of a sieve will result in incomplete separation with errors in the test. The following tabulation will be used as a guide in obtaining a minimum-weight sample:

Maximum Particle Size	Minimum Weight of Sample, g
3 in.	64,000 g
2 in.	19,000 g
1-1/2 in.	8,000 g
1 in.	2,400 g
3/4 in.	1,000 g
1/2 in.	300 g
3/8 in.	150 g
No. 4	50 g

† U.S. Army Engineer Waterways Experiment Station, CE, Sieve Analyses of Granular Soils by Division Laboratories, Engineering Study 516 (Vicksburg, Miss., October 1963).

‡ Clay shale materials require special preparation. See paragraph 5.

If the sample contains more than about 10% of sizes larger than the No. 4 sieve, it is generally advisable to separate the material on the No. 4 sieve, retaining both fractions for independent sieve analysis as subsequently described. If the sample contains plastic fines which tend to form hard lumps or to coat the coarser particles during air-drying, the entire sample should be placed in a pan filled with water and allowed to soak until all the soil lumps or the coatings have disintegrated, before it is separated on the No. 4 sieve. The coarser fraction and the fraction passing the No. 4 sieve including the fines and water should be retained for independent sieve analysis as subsequently described.

d. Procedure. (1) Material predominantly finer than the No. 4 sieve. The procedure for samples predominantly finer than the No. 4 sieve consists of the following steps:

(a) Record all identifying information for the sample, such as project, boring number, or other pertinent data, on a data sheet (see Plate V-1 for suggested form).

(b) Oven-dry the sample at 110 ± 5 C, allow to cool, and weigh. If the sample weighs less than 500 g, weigh it to the nearest 0.1 g; if the sample weighs over 500 g, weigh to the nearest 1 g. Record the dry weight of the sample on the data sheet.

(c) If the sample consists of clean sands or gravels, proceed with step (f).† If the sample contains plastic fines which tend to form hard lumps or to coat the coarser particles during oven-drying, place the oven-dry sample in a pan filled with enough water to cover all the material and allow it to soak until all the soil lumps or coatings have disintegrated. The length of time required for soaking will vary from about 2 to 24 hr, depending in general on the amount and plasticity of the fines.

(d) Transfer the sample and water from the pan to a No. 200 sieve, or if the sample contains an appreciable amount of coarse

† If there is any doubt concerning the cleanness of a sand or gravel, i.e. whether or not the particles may be coated with fines, or if the test is performed to determine whether or not a material complies with specifications, then the sample should be treated as subsequently described in steps (c) through (e).

particles, to a combined set of No. 4 and No. 200 sieves. Care should be taken not to overload the No. 200 sieve; if necessary, transfer the sample in increments. Wash the sample thoroughly through the sieves, discarding the material passing the No. 200 sieve. Larger particles in the sample may be individually washed and removed from the sieves.

(e) Oven-dry the combined material retained on the No. 4 and the No. 200 sieves and, after the sample has cooled, weigh. Record on the data sheet in the "Weight Retained in grams" column the difference between the original oven-dry weight and the oven-dry weight after washing. Use the washed sample for the remainder of the analysis.

(f) Select a nest of sieves suitable to the soil being tested. The choice of sieves usually depends on experience and judgment, and the use for which the grain-size curve is intended. Select as the top sieve. one with openings slightly larger than the diameter of the largest particle in the sample. Arrange the nest of sieves according to size as shown in Figure 1, with decreasing openings from top to bottom. Attach the bottom pan to the bottom of the smallest sieve used. Place the sample on the top sieve of the nest as shown in Figure 2 and put the cover plate over the top sieve.



Figure 1. Arrangement of sieves for grain-size analysis

(g) Place the nest of sieves in the shaking machine as shown in Figure 3 and shake them for 10 min, more or less, or until additional shaking does not produce appreciable changes in the amounts of material on each sieve. If a shaking machine is not available, the nest of sieves may be shaken by hand. In the hand operation, shake the nest of

sieves with a lateral and vertical motion, accompanied by jarring, to keep the material moving continuously over the surfaces of the sieves. Jarring is accomplished by occasionally dropping the nest lightly on several thicknesses of magazines. The nest should not be broken to rearrange particles or to manipulate them through a sieve by hand. Hand-shaking should be continued for at least 15 min.

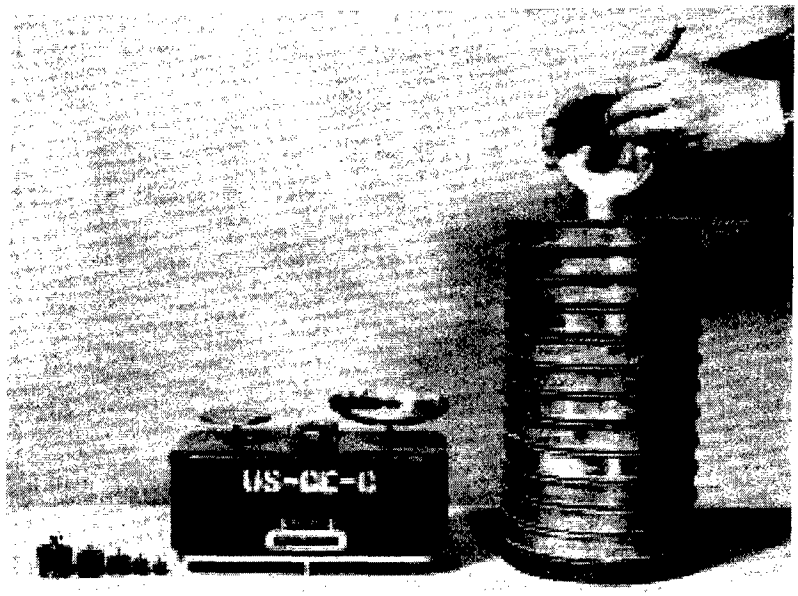


Figure 2. Placing soil on sieves

(h) Remove the nest of sieves from the mechanical shaker, if used. Beginning with the top sieve, transfer the contents of the sieve to a piece of heavy paper approximately 1 ft square. Carefully invert the sieve on the paper and gently brush the bottom of the sieve, as shown in Figure 4, to remove all the sample. Transfer the sample from the paper to the balance and weigh in accordance with requirements in

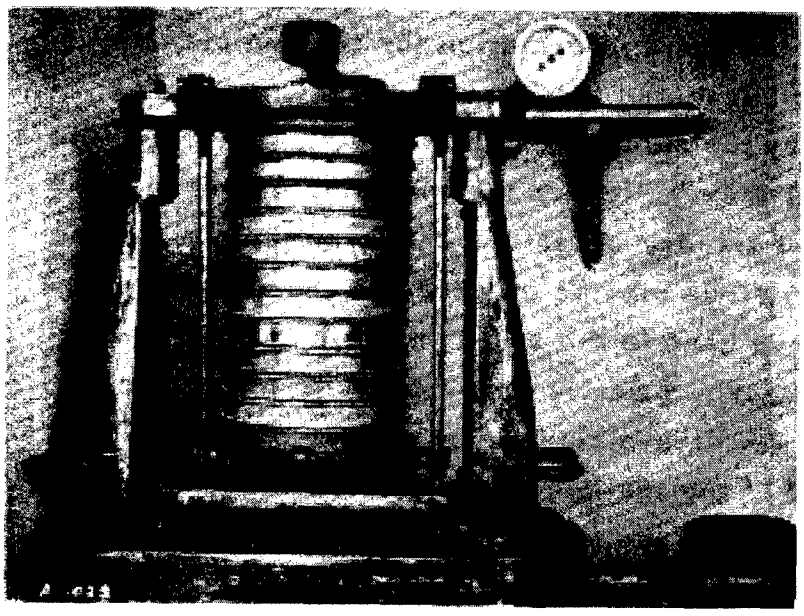


Figure 3. Nest of sieves placed in typical machine for shaking



'Figure 4. Removing soil from sieves

step (b). Care should be exercised that no loss of material occurs during the transfer. Coarser fractions may be transferred more readily from the sieves directly onto the balance pan. Record the weight of material retained on each sieve on the data sheet.

(i) Repeat step (h) for each sieve. The sum of the weights retained on each sieve and pan should equal the initial total weight of the sample within 1 percent. If the difference is greater than 1 percent, the sieving should be repeated.

(2) Material split on No. 4 sieve. The procedure for samples which have been split on the No. 4 sieve consists of the following steps:

(a) Record pertinent information for the sample on a data sheet (see Plate V-i for suggested form).

(b) Oven-dry the sample, allow it to cool, and weigh the fraction retained on the No. 4 sieve. Record the oven-dry weight on the data sheet. Alternatively, the air-dry weights of the total sample and the fraction retained on the No. 4 sieve may be utilized and the air-dry material retained on the No. 4 sieve used in the sieve analysis as in step (c) below. In the latter procedure, the relative percentages of materials greater than the No. 4 sieve are determined on an air-dry basis. This method is satisfactory provided the air-dry water contents of the plus and minus No. 4 portions of the sample are approximately equal.

(c) Proceed as in paragraphs 2d(1)(f) through 2d(1)(i). In general, it is advisable to use large sieves and a Ty-Lab or Gilson shaker for the coarse fraction.

(d) If the sample has not been washed during the preliminary treatment, process the material passing the No. 4 sieve according to paragraphs 2d(1)(b) through 2d(1)(i). If the material has been washed as part of the preliminary treatment, proceed with paragraphs 2d(1)(d) through 2d(1)(i), except that the material passing the No. 200 sieve in paragraph 2d(1)(d) should be oven-dried and weighed. This weight is added to the oven-dry weight of the plus No. 200 material to obtain the total weight of sample.

e. Computations. The percentage of material by weight retained on the various sieves is computed as follows:

$$\text{Percent retained} = \frac{\text{weight in g retained on a sieve}}{\text{total weight in g of oven-dry sample}} \times 100$$

If the sample has been split on the No. 4 sieve during preliminary treatment and the air-dried coarser fraction sieved independently, the percent retained for the coarser fraction is computed as follows:

$$\text{Percent retained} = \frac{\text{air-dry weight in g retained on a sieve}}{\text{air-dry weight in g of total sample}} \times 100$$

Similarly, for the finer fraction when oven-dry weights are used:

$$\text{Percent retained} = \frac{\text{weight in g retained on a sieve}}{\text{oven-dry weight in g of sample passing No. 4 sieve}} \times \text{percent passing No. 4}$$

where the percentage passing No. 4 sieve is computed on an air-dry basis. The values of percent retained based on the above formulas refer to the total weight of sample. Computation of a partial percent retained as indicated in Plate V-i is necessary only when the sample is initially separated on the No. 200 sieve for purposes of a combined analysis, as subsequently described. The cumulative percent finer by weight than an individual sieve size (percent finer) is calculated by subtracting the percent retained

on the individual sieve from the cumulative percent finer than the next larger sieve.

f. Presentation of Results. The results of the sieve analysis are presented in the form of a grain-size distribution curve on a semi-logarithmic chart as shown in Plate V-2. The grain-size distribution curve is obtained by plotting particle diameter (sieve opening) on the abscissa (logarithmic scale) and the percent finer by weight on the ordinate (arithmetic scale).

3. HYDROMETER ANALYSIS. a. Description. The hydrometer method of analysis is based on Stokes' law, which relates the terminal velocity of a sphere falling freely through a fluid to the diameter. The relation is expressed according to the equation:

$$v = \frac{\gamma_s - \gamma_f}{1800\,\eta} D^2$$

where v = terminal velocity of sphere, cm per sec
γ_s = density of sphere, g per cm^3
γ_f = density of fluid, g per cm^3
η = viscosity of fluid, g-sec per cm^2
D = diameter of sphere, mm

It is assumed that Stokes' law can be applied to a mass of dispersed soil particles of various shapes and sizes. The hydrometer is used to determine the percentage of dispersed soil particles remaining in suspension at a given time. The maximum grain size equivalent to a spherical particle is computed for each hydrometer reading using Stokes' law. The hydrometer analysis is applicable to soils passing the No. 10 sieve for routine classification purposes; when greater accuracy is required (such as in the study of frost-susceptible soils), the hydrometer analysis should be performed on only the fraction passing the No. 200 sieve (see paragraph COMBINED ANALYSIS).

b. Apparatus. The apparatus should consist of the following:

(1) Hydrometer, calibrated at 20/20 C (68/68 F), graduated in specific gravity or grams per liter with a range of 0.995 to 1.040 and 0 to 50, respectively. The accuracy of the specific gravity hydrometer shall be ± 0.001 and of the gram-per-liter hydrometer, ±1.

(2) Dispersion apparatus, either of two types may be used:

(a) A mechanically operated stirring device in which a suitably mounted electric motor turns a vertical shaft at a speed of not less than 10,000 rpm without load. The shaft shall be equipped with a replaceable stirring paddle of metal, plastic, or hard rubber. Details of a typical paddle are shown in Figure 5. A special dispersion cup conforming to either of the designs shown in Figure 5 shall be provided to hold the sample while it is being dispersed.

(b) An air dispersion device such as the air -jet dispersion tube device

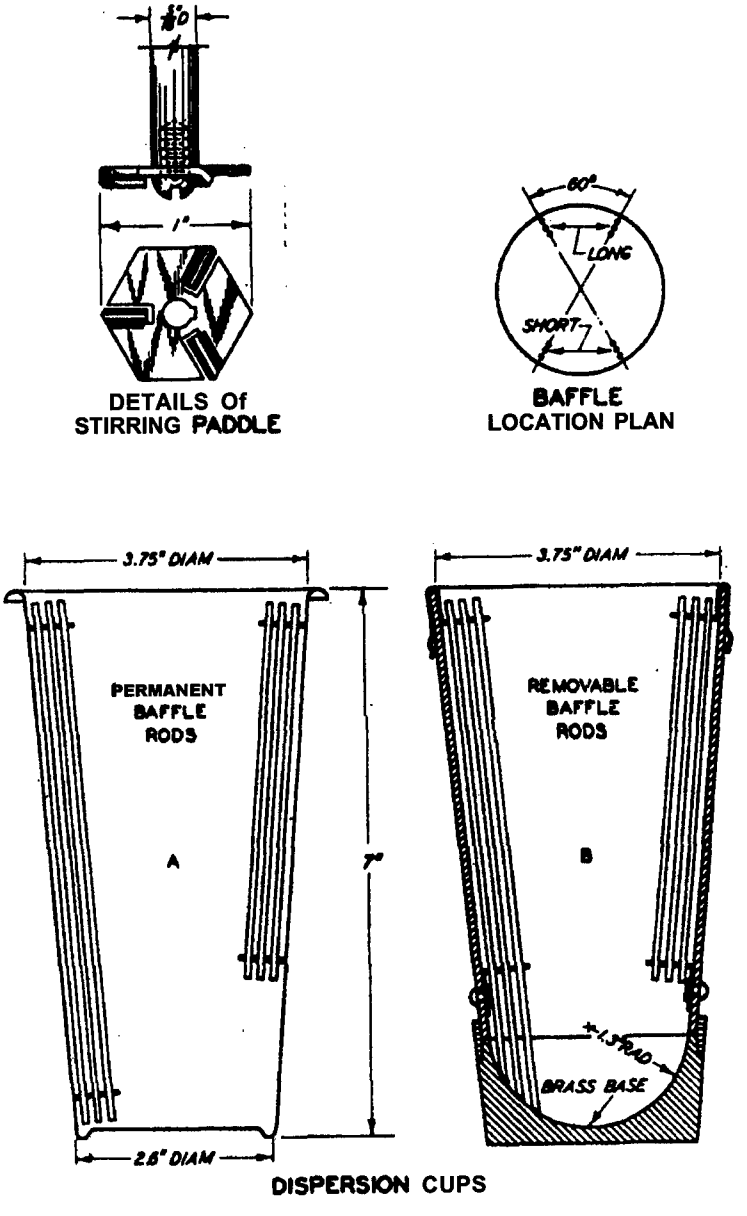


Figure 5. Detail of stirring paddle and dispersion cups

developed at Iowa State College.†

(3) Sedimentation cylinder, of glass, essentially 18 in, high and 2-1/2 in. in diameter and marked for a volume of 1000 ml.

(4) Centigrade thermometer, range 0 to 50 C, accurate to 0.5 c.

(5) Timing device, a watch or clock with a second hand.

(6) Balance, sensitive to 0.1 g.

(7) Oven (see Appendix I', WATER CONTENT - GENERAL).

c. Hydrometer Calibration. The hydrometer shall be calibrated‡ to determine its true depth in terms of the hydrometer reading (see Fig. 6) in the following steps:.

(1) Determine the volume of the hydrometer bulb, V_R. This may be determined in either of two ways:

(a) By measuring the volume of water displaced. Fill a 1000-cc graduate with water to approximately 700 cc. The water should be at about 20 C. Observe and record the reading of the water level. Insert the hydrometer and again observe and record the reading. The difference in these two readings equals the volume of the bulb plus the part of the stem that is submerged. The error due to inclusion of this latter quantity is so small that it may be neglected for practical purposes.

(b) By determining the volume from the weight of the hydrometer. Weigh the hydrometer to 0.01 g on the laboratory balance. Since the specific gravity of a hydrometer is, about unity, the weight in grams may be recorded as the volume in cubic centimeters. This volume includes the volume of the bulb plus the volume of the stem. The error

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† T. Y. Chu and D. T. Davidson, "Simplified air-jet dispersion apparatus. for mechanical analysis of soils," Proceedings, Highway Research Board, vol. 32 (1953), pp. 541-547.

‡ ASTM hydrometers 151 H or 152 H (ASTM Designation: E 100) have a uniform size; therefore, only a single calibration is required, which can be applied to all ASTM hydrometers of this type.

due to inclusion of the stem volume is negligible.

(2) Determine the area, A, of the graduate in which the hydrometer is to be used by measuring the distance between two graduations. The area, A, is equal to the volume included between the graduations divided by the measured distance.

(3) Measure and record the distances from the lowest calibration mark on the stem of the hydrometer to each of the other major calibration marks, R.

(4) Measure and record the distance from the neck of the bulb to the lowest calibration mark. The distance, H_1, corresponding to a reading, R, equals the sum of the two distances measured in steps (3) and (4).

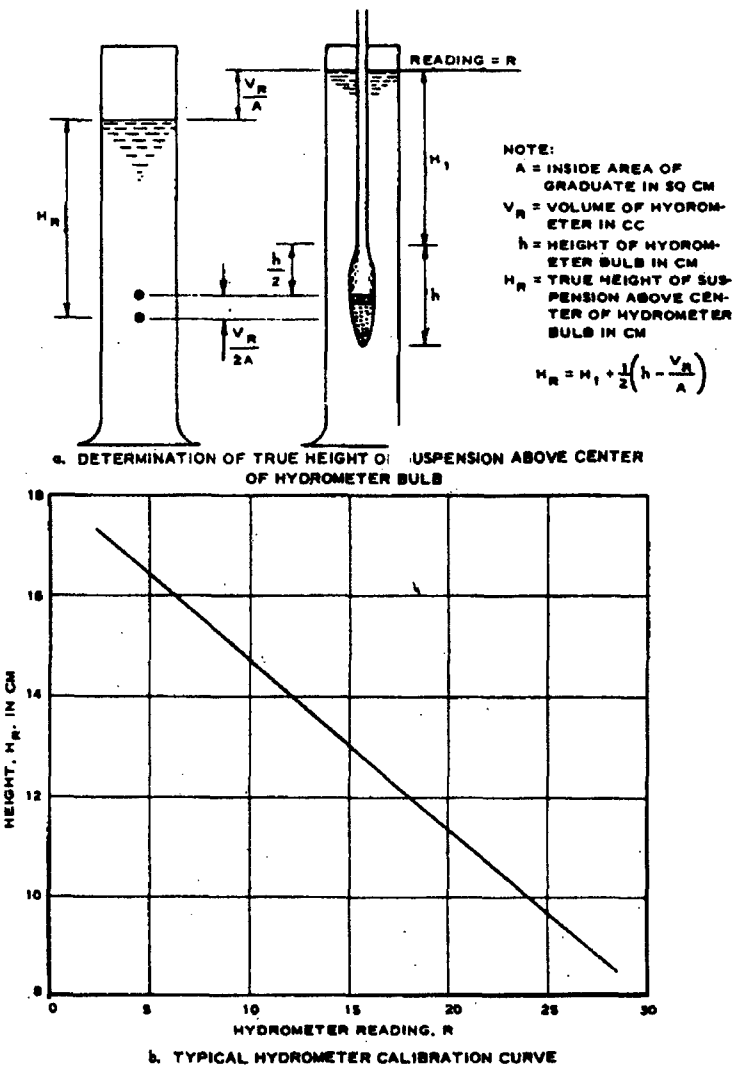


Figure 6. Hydrometer calibration

(5) Measure the distance from the neck to the tip of the bulb. Record this as h, the height of the bulb. The distance, h/2, locates the center of volume of a symmetrical bulb. If a nonsymmetrical bulb is used, the center of volume can be determined with sufficient accuracy by projecting the shape of the bulb on a sheet of paper and locating the center of gravity of this projected area.

(6) Compute the true distances, HR, corresponding to each

of the major calibration marks, R, from the formula:

$$H_R = H_1 + \frac{1}{2}\left(h - \frac{V_R}{A}\right)$$

(7) Plot the curve expressing the relation between HR and R as shown in Figure 6. The relation is essentially a straight line for hydrometers having a streamlined shape.

d. Meniscus Correction. Hydrometers are calibrated to read correctly at the surface of the liquid. Soil suspensions are not transparent and a reading at the surface is not possible; therefore, the hydrometer reading must be made at the upper rim of the meniscus. The meniscus correction, C_m, which is a constant for a given hydrometer, is determined by immersing the hydrometer in distilled or demineralized water and observing the height to which the meniscus rises on the stem above the water surface, For most hydrometers it will be found that C_m is equal to approximately 0.5, and this value can be assumed for routine testing.

e. Preparation of Sample. The approximate size of sample to be used for the hydrometer analysis varies according to the type of soil being tested, as shown in the tabulation below:

Soil Type	Dry Weight, g
Fat clays	30
Lean clays and silty soils	50
Sandy soils	75†

† Up to 150 g of sandy soil can be used for the hydrometer analysis provided no more than 50 g of the sample is finer than the No. 200 sieve.

The exact dry weight of the sample in suspension may be determined either before or after the test. However, oven-drying some clays before the test may cause permanent changes in the apparent grain sizes. Samples of such soils should, if possible, be preserved at the natural

water content and tested without first being oven-dried, the dry weight either being obtained after the hydrometer analysis or computed according to the formula:

$$\text{Dry weight of specimen} = \frac{\text{weight of wet soil}}{1 + \frac{\text{water content}}{100}}$$

$$W_s = \frac{W}{1 + 0.01\ w}$$

w having been determined on an untested portion of the sample. Furthermore, if samples are dried and weighed before the test, any loss of material during the test will affect the results.

f. Dispersing Agent. Very fine soil grains in a suspension normally will tend to flocculate, i.e. to adhere with sufficient force that they settle together. Consequently, a dispersing agent to prevent flocculation of the soil grains during the test should be added to all samples. The following dispersing agents, listed in approximate order of effectiveness, have been found to be satisfactory for most types of soils.†

	Dispersing Agents	Stock Solution: Concentration	Stock Solution: g per liter	Manufacturer
1	Sodium tripolyphosphate	0.4 N	29	Blockson Chem. Co., Joliet, Ill.
2	Sodium polyphosphate	0.4 N	36	Blockson Chem. Co., Joliet, Ill.
3	Sodium tetraphosphate (trade name "Quadrafos")	0.4 N	31	Rumford Chem. Works, Rumford, R. I.
* 4	Sodium Hexametaphosphate (sometimes called sodium metaphosphate) adjusted to pH8-9 with Na_2CO_3	0.4 N	41	Most laboratory chemical supply c os.

The chemical product Calgon available in grocery stores shall not be used as a dispersing agent as it no longer contains sodium hexametaphosphate. Sodium silicate shall not be used as a dispersing agent since it gives unsatisfactory dispersion while at the same time permitting flocculation to a

† A. M. Wintermyer and E. B. Kinter, "A study of dispersing agents for particle-size analysis of soils," Public Roads, vol. 28, No. 3 (August 1954), pp 55-62.

point where it is not apparent to visual examination. Phosphate solutions are somewhat unstable and therefore should not be stored for extended periods of time. In most instances, 15 ml of a dispersing agent solution is adequate. However, should flocculation tend to continue, a second or third addition of 15 ml of solution may be added.

The addition of a dispersing agent to the soil suspension results in an increase in density of the liquid and necessitates a correction to the observed hydrometer reading. The correction factor, C_d, is determined by adding to a 1000-ml graduate partially filled with distilled or demineralized water the amount of dispersing agent to be used for the particular test, adding additional distilled water to the 1000-ml mark, then inserting a hydrometer and observing the reading. The correction factor, C_d, is equal to the difference between this reading and the hydrometer reading in pure distilled or demineralized water.

The addition of a dispersing agent also increases the weight o solids in the suspension. If the oven-dry weight of soil used for the hydrometer analysis is obtained at the end of the test, this weight must be corrected by subtracting the dry weight of the dispersing agent used.

g. Procedure. The procedure shall consist of the following steps:

(1) Record all identifying information for the sample, such as project, boring number, or other pertinent data, on a data sheet (see Plate V-3 for suggested form).

(2) Determine the dispersing agent correction, C_d, and the meniscus correction, C_m, unless they have been previously established. Record this information on the data sheet.

(3) Determine *or* estimate the specific gravity of solids and record on the data sheet.

(4) If the oven-dry weight is to be obtained at the start of the test, oven-dry the sample, allow to cool, and weigh to nearest 0.1 g. Record the dry weight on the data sheet. Place the sample in a numbered dish and add distilled or demineralized water until the sample is submerged. Add

the dispersing agent at this time. Allow the sample to soak overnight or until all soil lumps have disintegrated. Highly organic soils require special treatment, and it may be necessary to oxidize the organic matter in order to perform a hydrometer analysis on these soils. Oxidation is accomplished by mixing the sample with a solution of 30 percent hydrogen peroxide; this solution will oxidize all the organic matter. If only small amounts of organic matter are present, treatment with hydrogen peroxide may be omitted.

(5) Transfer the soil-water slurry from the dish to a dispersion cup (Fig. 5), washing† any residue from the dish with distilled or demineralized water. Add distilled water to the dispersion cup, if necessary, until the water surface is 2 or 3 in. below the top of the cup; if the cup contains too much water, it will splash out while mixing. Place the cup in the dispersing machine and disperse the suspension for 1 to 10 min. The lower the plasticity of the soil the shorter the time required to disperse it in the cup.‡

(6) Transfer the suspension into a 1000-ml sedimentation cylinder and add distilled or demineralized water until the volume of the uspension equals 1000 ml. The suspension should be brought to the temperature expected to prevail during the test.

(7) One minute before starting the test, take the graduate in one hand and, using the palm of the other hand or a suitable rubber cap as a stopper, shake the suspension vigorously for a few seconds in order to transfer the sediment on the bottom of the graduate into a uniform suspension. Continue the agitation for the remainder of the minute by turning the cylinder upside down and back. Sometimes it is necessary to loosen the

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† A large syringe or wash-water bottle is a convenient device for. handling the water in the washing operation.

‡ Air dispersion may be used in place of mechanical dispersion. A dispersion time of 10 min is recommended, using an air pressure of 25 psi for clays and silts and 10 psi for sands. Several comparative tests indicate that the air dispersion apparatus gives a higher degree of dispersion of clayey soils while causing less degradation of sands than the mechanical stirring apparatus. See: Chu and Davidson, op. cit., and U. S. Bureau of Reclamation, Comparison of Dispersion Methods for Soil Gradation Analysis< Earth Laboratory Report No. EM-618 (Denver, Colo., May 1961).

sediment at the bottom of the cylinder by means of a glass rod before shaking. Alternatively, the suspension may be agitated by means of a hand agitator for one minute prior to testing. A schematic drawing of a hand agitator is shown in Figure 7. A uniform distribution of the soil particles in the suspension is accomplished by moving the hand agitator up and down through the suspension for one minute. This process also prevents the accumulation of sediment on the base and sides of the graduate.

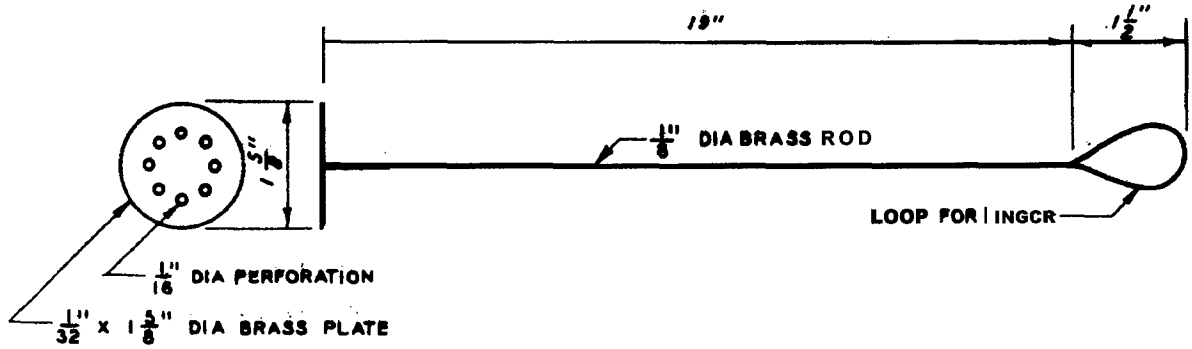


Figure 7. Hand agitator for hydrometer cylinder

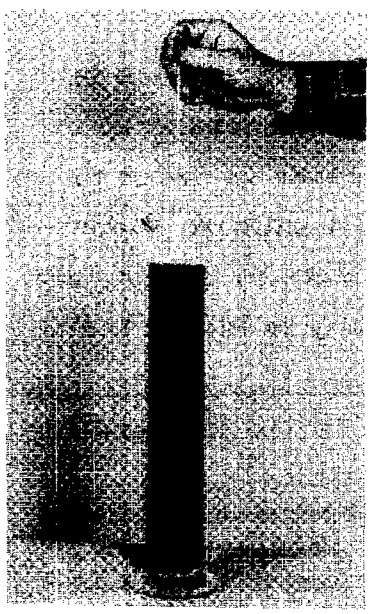


Figure 8. Immersing hydrometer in suspension prior to making observation

(8) At the end of 1 min, set the cylinder on a table. If foam is present, remove it from the top of the suspension by lightly touching it with a piece of soap. Slowly immerse the hydrometer in the liquid 20 to 25 sec before each reading, as shown in Figure 8. Care should be exercised when inserting and removing the hydrometer to prevent disturbance of the suspension.

(9) Observe and record the hydrometer readings on the data sheet after 1 and 2 min have elapsed from the time the cylinder is placed on the table. As soon as the 2-min reading has been taken, carefully remove the hydrometer from the suspension and place it in a graduate of clean water. (If a hydrometer is left in a soil suspension for any length of time, material will

settle on or adhere to the hydrometer bulb and this will cause a significant error in the reading,) Again insert the hydrometer in the suspension and record readings after elapsed times of 4, 15, 30, 60, 120,† 240, and 1440 min, removing the hydrometer from the suspension after each reading and placing it in a graduate of clean water. Make all hydrometer readings at the top of the meniscus. For hydrometers graduated to read in specific gravity of the suspension, read only the last two figures and estimate the third. Record the indicated specific gravity, minus 1, multiplied by 1000 (example: the reading 1.0225 should be recorded as 22.5). For hydrometers graduated to read grams per liter of suspension, record the actual reading.

(10) At the end of 2 min and after each subsequent hydrometer reading, place a thermometer in the suspension and record the temperature reading on the data sheet. The temperature shall be recorded to ±0.5 C. Temperature changes of the soil suspension during the test will affect the test results. Variations in temperature should be minimized by keeping the suspension away from heat sources such as radiators, sunlight, or open windows. A constant-temperature bath provides a convenient means of controlling temperature effects.

(11) If the dry weight of the sample is to be obtained at the end of the test, carefully wash all the suspension into an evaporating dish. Oven-dry the material, allow to cool, and determine the sample weight. Subtract the dry weight of dispersing agent used from this weight to obtain the oven-dry weight of soil.

h. Computations. (1) Corrected hydrometer reading. Compute the corrected hydrometer readings, R, for use in computing particle diameter by adding the meniscus correction, C_m, to the actual hydrometer readings, R'. Record the corrected reading, R, on the data sheet.

(2) Computation of particle diameter. Calculate the particle

† A final reading after 120 min is sufficient for most soils when hydrometer analysis is used for classification purposes.

diameter corresponding to a given hydrometer reading on the basis of Stokes' equation, using the nomograph shown in Figure 9. The R-scale corresponding to the distances H_R is prepared using the hydrometer calibration curves as shown in Figure 6. The R-scale shall be designed for the particular hydrometer used in the test. A key showing the steps to follow in computing D for various values of R is shown on the chart. Record the particle diameters, D, on the data sheet.

(3) Percent finer. To compute the percent of particle diameters finer than that corresponding to a given hydrometer reading, subtract the dispersing agent correction, C_d, from the corrected hydrometer reading, R. A temperature correction factor, m, must also be added algebraically to each of the readings. This factor can be either positive or negative depending on the temperature of the suspension at the time of each reading. Obtain the temperature correction factors from Table V-i and record them on the data sheet. Record the values of $R - C_d + m$ on the data sheet. The $R - C_d$ t n values are used to compute percent finer according to the following formulas:

Hydrometer calibrated in specific gravity:

$$\text{Percent finer by weight} = \frac{G_s}{G_s - 1} \times \frac{100}{W_s} (R - C_d + m)$$

Hydrometer calibrated in grams per liter:

$$\text{Percent finer by weight} = \frac{100}{W_s} (R - C_d + m)$$

where G_s = specific gravity of solids

$W_s = W_o$ = oven-dry weight in g of soil used for hydrometer analysis

$R - C_d + m$ = corrected hydrometer reading minus dispersing agent correction plus, algebraically, temperature correction

Calculations for routine work can be greatly facilitated by using charts, tables, and other simplifying aids based on a given oven-dry weight of the sample and average specific gravity values for the major soil groups.

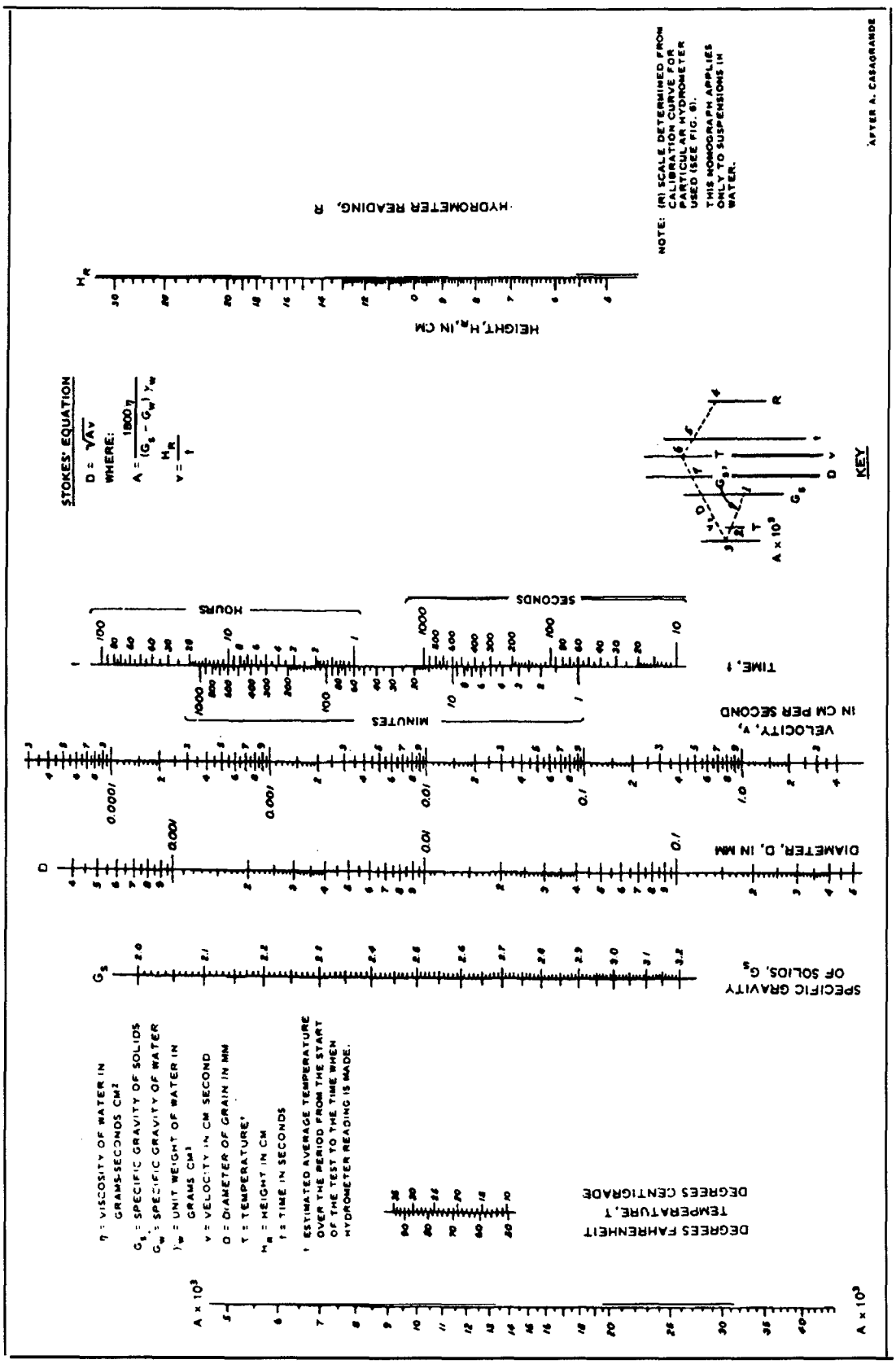


Figure 9. Nomographic chart

Table V-1

Temperature Correction Factor, m, for Use in Computing Percent Finer

Degrees C	Degrees F	Correction	Degrees C	Degrees F	Correction
14.0	57.2	-0.9	24.0	75.2	+0.8
14.5	58.1	-0.8	24.5	76.1	+0.9
15.0	59.0	-0.8	25.0	77.0	+1.0
15.5	59.9	-0.7	25.5	77.9	+1.1
16.0	60.8	-0.6	26.0	78.8	+1.3
16.5	61.7	-0.6	26.5	79.7	+1.4
17.0	62.6	-0.5	27.0	80.6	+1.5
17.5	63.5	-0.4	27.5	81.5	+1.6
16.0	64.4	-0.4	28.0	82.4	+1.8
18.5	65.3	-0.3	28.5	83.3	+1.9
19.0	66.2	-0.2	29.0	84.2	+2.1
19.5	67.1	-0.1	29.5	85.1	+2.2
20.0	68.0	0.0	30.0	86.0	+2.3
20.5	68.9	+0.1	30.5	86.9	+2.5
21.0	69.8	+0.2	31.0	87.8	+2.6
21.5	70.7	+0.3	31.5	88.7	+2.8
22.0	71.6	+0.4	32.0	89.6	+2.9
22.5	72.5	+0.5	32.5	90.5	+3.0
23.0	73.4	+0.6	33.0	91.4	+3.2
23.5	74.3	+0.7	33.5	92.3	+3.3
			34.0	93.2	+3.5

1. Presentation of Results. The data obtained from the hydrometer analysis are presented in the form of a grain-size distribution curve on a semilogarithmic chart, as shown in Plate V-2.

4. COMBINED ANALYSIS. a. Description. A combined analysis is necessary for soils containing material finer than the U. S. Standard No. 200 sieve when the grain-size distribution of the material passing the No. 200 sieve is of interest. A sieve analysis is performed on the material retained on the No. 200 sieve, and a hydrometer test is performed on the material passing the No. 200 sieve.

b. Apparatus. The apparatus for the combined analysis is the same as that used for both the hydrometer and sieve analyses.

c. Preparation of Sample. A representative sample for the combined analysis is selected and prepared in the manner described in paragraph 2c. The total amount of sample should be sufficient to yield required amounts of material for both the sieve and hydrometer analyses. A visual inspection of the sample will usually suffice to indicate the need for intermediate steps such as large screen processing for the plus No. 4 fraction, washing, etc. Samples of soils having fines with little or no plasticity are oven-dried, weighed, and then separated on the No. 200 sieve. The plus and minus No. 200 sieve fractions are preserved for the sieve and hydrometer analyses, respectively.

Soils containing plastic fines may also be oven-dried initially. However, if the sample contains plastic fines which tend to form hard lumps or to coat the coarser particles during oven-drying, the sample is placed in a pan filled with enough water to cover all the material and allowed to soak until all the lumps or coatings have been reduced to individual particles. The length of time required for soaking will vary from 2 to 24 hr, depending in general on the amount and plasticity of the fines, The water and soil mixture is then washed over a No. 200 sieve (and No. 4 sieve, if necessary), The coarser fractions are preserved for a sieve analysis, and the soil and water passing the No. 200 sieve are preserved

for a hydrometer analysis. Excess water with the lines is removed by evaporation, filtration, or wicking. If the grain size of the plastic fines would be altered by oven-drying. The oven-dry weight of the fines is determined after the hydrometer test.

In routine testing when all soil particles are finer than the No. 10 sieve size, the hydrometer test may be performed on a total sample of known dry weight; the sample is then washed through the No. 200 sieve, and finally the sieve analysis is performed on the oven-dried fraction retained on the No. 200 sieve.

d. Procedure. The procedure shall consist of the following steps:

* (1) Record identifying information for the sample on both the sieve and hydrometer analysis data sheets (see Plate V-1). *

(2) Perform a sieve analysis on a representative portion of the sample retained on the No. 200 sieve, using the procedures described in paragraphs 2d(1) and 2d(2).

(3) Perform a hydrometer analysis on a portion (see paragraph 3e for approximate weight) of the sample passing the No. 200 sieve, using the procedure described in paragraph 3g.

e. Computations. The computations consist of the following steps:

(1) Compute the percentage retained on the No. 200 sieve for the total sample used in the combined analysis as follows:

$$\text{Percent retained on No. 200 sieve} = \frac{W_1}{W_S} \times 100$$

where W_1 = dry weight of sample retained on No. 200 sieve

W_S = total dry weight of sample used for combined analysis

(2) Compute the data from the sieve analysis in the same manner as outlined in paragraphs 2d(1) and 2d(2), except that the percent retained for each sieve shall be based only on that portion of the total material used for the sieve analysis. As the amount of material used in the sieve analysis may be less than W_1, it will be necessary to compute

a partial percent retained as follows:

$$\text{Partial percent retained} = \frac{\text{weight in g retained on a sieve}}{\text{total weight in g of oven-dry sample used for sieve analysis}} \times 100$$

The total percent retained is computed as follows:

$$\text{Total percent retained} = \text{partial percent retained} \times \frac{W_1}{W_s}$$

The total percent finer is computed as follows:

$$\text{Total percent finer} = 100 - \text{total percent retained}$$

(3) Compute the data from the hydrometer analysis in the same manner as outlined in paragraphs 3h(1) through 3h(3), except that the results shall be shown in terms of a partial percent finer. As in the sieve analysis, the amount of material used for the hydrometer analysis may be less than $W_s - W_1$, therefore a partial percent finer is computed as follows:

Hydrometer calibrated in specific gravity:

$$\text{Partial percent finer} = \frac{G_s}{G_s - 1} \times \frac{100}{W_o}(R - C_d + m)$$

Hydrometer calibrated in grams per liter:

$$\text{Partial percent finer} = \frac{100}{W_o}(R - C_d + m)$$

where W_o = oven-dry weight in g of soil used for hydrometer analysis

Other terms were defined previously.

The total percent finer is computed as follows:

$$\text{Total percent finer} = \text{partial percent finer} \times \frac{W_s - W_1}{W_s}$$

f. Presentation of Results. The results of the combined analysis

in terms of particle diameter and total percent finer by weight are presented in the form of grain-size distribution curves on a semilogarithmic chart as shown in Plate V-2. The curves obtained from the sieve and hydrometer analyses are joined by constructing a smooth curve between them.

5. PROCEDURES FOR PREPARING CLAY SHALE MATERIAL. The procedures for preparing clay shale material shall be the same as those described in paragraph 4, page III- 14, Appendix III, LIQUID AND PLASTIC LIMITS. Material for a particle-size distribution test should be removed from a processed batch and the test performed in accordance with the procedures described in this appendix. However, the material should not be oven-dried before testing, and the hydrometer analysis should be of duration sufficient to determine the percent finer than 2-μ size.

6. POSSIBLE ERRORS. Following are possible errors that would cause inaccurate determinations of grain-size distribution:

a. Sieve Analysis. (1) Aggregations of particles not thoroughly broken. If the material contains plastic fines, the sample should be slaked before sieving.

(2) Overloading sieves. This is the most common and most serious error associated with the sieve analysis and will tend to indicate that a material is coarser than it actually is. Large samples may have to be sieved in several portions, and the portions retained on each sieve recombined afterwards for weighing.

(3) Sieves shaken for too short a period or with inadequate horizontal or jarring motions. The sieves must be shaken so that each particle is exposed to the sieve openings with various orientations and has every opportunity to fall through.

(4) Broken or deformed sieve screens. Sieves must be frequently inspected to ensure they contain no openings larger than the standard.

(5) Loss of material when removing soil from each sieve.

b. Hydrometer Analysis. (1) Soil oven-dried before test. Except

for inorganic soils of low dry strength, oven-drying may cause permanent changes in the particle sizes.

(2) Unsatisfactory type or quantity of dispersing agent. Whenever new or unusual soils are tested, trials may be necessary to determine the type and quantity of chemical which gives the most effective dispersion and deflocculation.

(3) Incomplete dispersion of. soil into suspension.

(4) Insufficient shaking or agitating of suspension in cylinder at start of test.

(5) Too much soil in suspension. The results of the hydrometer analysis will be affected if the size of the sample exceeds the recommendations given in paragraph 3e.

(6) Disturbance of suspension while inserting or removing hydrometer. Such disturbance is most likely to result when the hydrometer is withdrawn too rapidly after a reading.

(7) Stem of hydrometer not clean. Dirt or grease on the stem may prevent full development of the meniscus.

(8) Nonsymmetrical heating of suspension,

(9) Excessive variation in temperature of suspensionduring test.

(10) Loss of material after test. If the oven-dry weight of the soil is obtained after the test, all of the suspension must be washed carefully from the cylinder.

c. Combined Analysis. (1) Insufficient washing of material over the No. 200 sieve. The dispersing agent should be added to the water in which the sample is soaked and the soil-water mixture should be frequently manipulated to aid the separation of particles; coarser particles may be removed from the mixture and washed free of fines by hand to reduce the quantity of material to be washed on the sieve. While the additional water used for washing should be held to a minimum, enough must be added to insure adequate removal of the fines.

(2) Loss of suspension passing the No. 200 sieve.

SIEVE AND HYDROMETER ANALYSIS

(EM 1110-2-1906)

PART I – SIEVE ANALYSIS	DATE
PROJECT	
BORING NO.	SAMPLING NO.
TOTAL WEIGHT IN GRAMS OF SAMPLE, W_S = _____	WEIGHT IN GRAMS OF MATERIAL >NO. 4 SIEVE = _____

STEVE OPENINGS		U.S. STANDARD SIEVE SIZE OR NUMBER	WEIGHT RETAINED IN GRAMS	PERCENT RETAINED		PERCENT FINER BY WEIGHT
INCHES	MILLIMETERS			PARTIAL	TOTAL	
3.00		3-in.				
2.00		2-in.				
1.50		1-1/2-in.				
1.00	25.4	1-in.				
0.750	19.1	3/4-in.				
0.500	12.7	1/2-in.				
0.375	9.52	3/8-in.				
0.250	6.35	No. 3				
0.187	4.76	No. 4				
		Pan				
0.132	3.36	No. 6				
0.094	2.38	No. 8				
0.079	2.00	No. 10				
0.047	1.19	No. 16				
0.033	0.84	No. 20				
0.023	0.59	No. 30				
0.0165	0.42	No. 40				
0.0117	0.297	No. 50				
0.0083	0.210	No. 70				
0.0059	0.149	No. 100				
0.0041	0.105	No. 140				
0.0029	0.074	No. 200				
		Pan				
TOTAL WEIGHT IN GRAMS						

Partial percent retained = $\frac{\text{wt in grams retained on a sieve}}{\text{wt in grams of sample used for a given series of sieves}} \times 100$

Total percent retained = $\frac{\text{wt in grams retained on a sieve}}{\text{total wt in grams of oven-dry sample}} \times 100$

For an individual sieve, the percent finer by weight = percent finer than next larger sieve – percent retained on individual sieve

REMARKS

TECHNICIAN _____ COMPUTED BY _____ CHECKED BY _____

TECHNICIAN	COMPUTED BY	CHECKED BY

PART II – HYDROMETER ANALYSIS			DATE
PROJECT			
BORING NO.:			
SAMPLE OR SPECIMEN NO.		CLASSIFICATION	
DISH NO.	GRADUATE NO.		HYDROMETER NO.
DISPERSING AGENT USED			QUANTIY
DISPERSING AGENT CORRECTION, C_d = _____		MENISCUS CORRECTION, C_m = _____	

TIME	ELAPSED TIME MIN	TEMP °C	HYDRO. READING (R')	CORRECTED READING (R)	PARTICLE DIAMETER (D), MM	TEMP CORRECTION (m)	$R - C_d + m$	PERCENT FINER	
								PARTIAL	TOTAL

WEIGHT IN GRAMS			
	DISH PLUS DRY SOIL		
	DISH		
	DRY SOIL	W_o	

Specific gravity of solids, G_s =

Corrected hydrometer reading (R) = hydrometer reading (R') + C_m

The particle diamter (D) is calculated from Stoke's equation using corrected hydrometer reading. Use nomographic chart for solution of Stocke's equation.

Hydrometer graduated in specific gravity

$$\text{Partial percent finer} = \frac{G_s}{G_s - 1} \times \frac{100}{W_o}(R - C_d + m)$$

Hydrometer graduated in grams per liter

$$\text{Partial percent finer} = \frac{100}{W_o}(R - C_d + m)$$

$$\text{Total percent finer} = \text{partial percent finer} \times \frac{W_s - W_1}{W_s}$$

W_s = total oven-dry wt of sample used for combined analysis

W_o = oven-dry wt in grams of soil used for hydrometer analysis

W_1 = oven-dry wt of sample retained on No. 200 sieve

REMARKS _____

TECHNICIAN	COMPUTED BY	CHECKED BY

Reverse of ENG Form 3841

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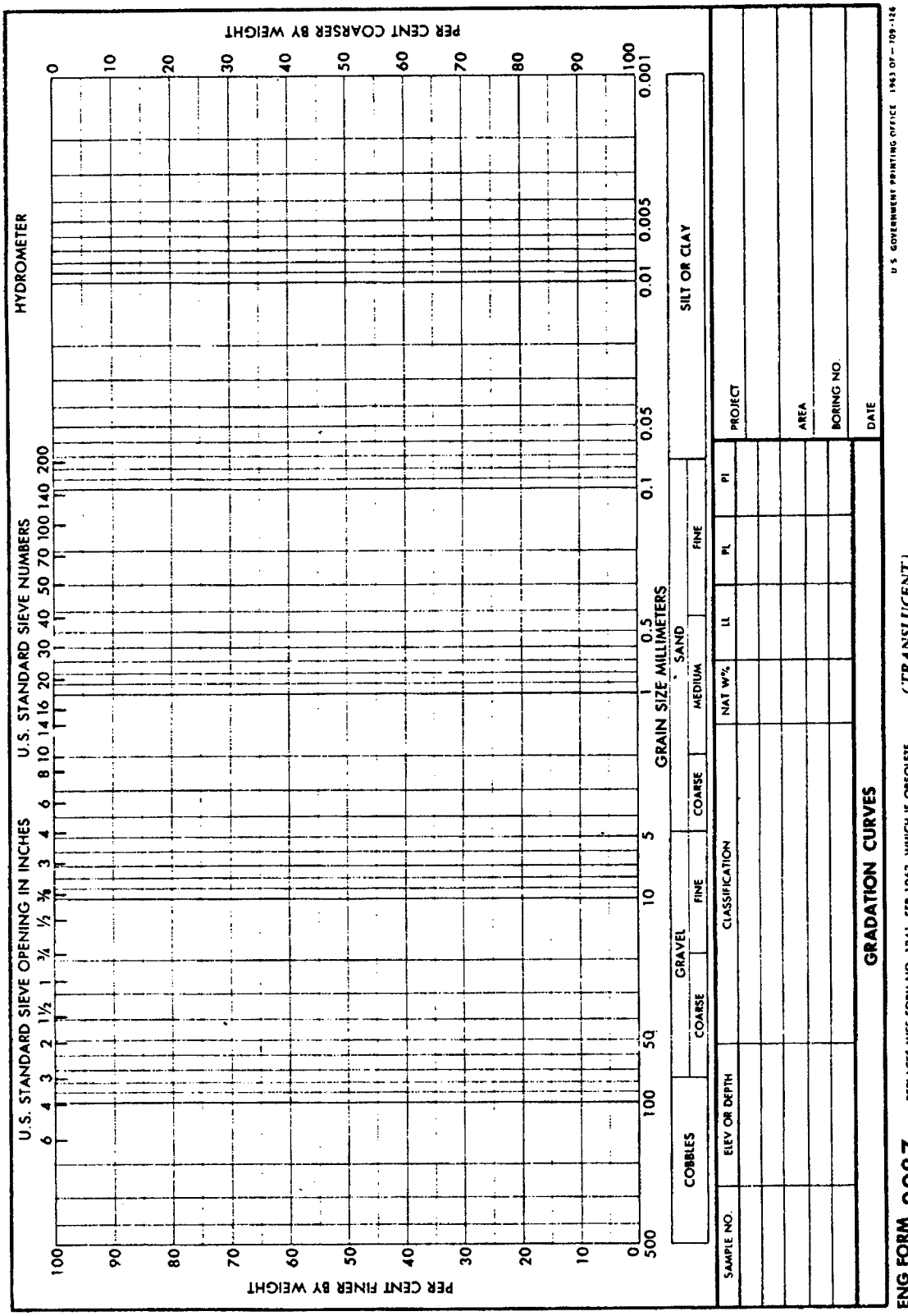

HYDROMETER
U.S. STANDARD SIEVE OPENING IN INCHES
U.S. STANDARD SIEVE NUMBERS
PER CENT FINER BY WEIGHT
PER CENT COARSER BY WEIGHT
GRAIN SIZE MILLIMETERS
COBBLES
GRAVEL
COARSE
FINE
SAND
COARSE
MEDIUM
FINE
SILT OR CLAY
SAMPLE NO.
ELEV OR DEPTH
CLASSIFICATION
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GRADATION CURVES
ENG FORM 2087
1 MAY 63
REPLACES WES FORM NO. 1241, SEP 1962, WHICH IS OBSOLETE.
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U.S. GOVERNMENT PRINTING OFFICE 1963 O-709-126

PLATE V-2 *

APPENDIX VI:
COMPACTION TESTS

1. INTRODUCTION. In the laboratory compaction test, a soil at a known water content is placed in a specified manner in a mold of given dimensions and subjected to a compactive effort of controlled magnitude after which the resulting unit weight of the soil is determined. The procedure is repeated at various water contents until a relation between water content and unit weight of the soil is established.

The laboratory compaction procedure is intended to simulate the compactive effort anticipated in the field. As a general rule the standard compaction test shall be used to simulate field compaction for routine foundation and embankment design. In special cases, to suit anticipated construction procedures, it may be necessary to use higher or lower compactive efforts on the soil. For a higher compactive effort the modified compaction test, and for a lower compactive effort the **15-blow** compaction test shall be used. Details of the standard, modified, and **15-blow** compaction tests are given below.

2. STANDARD COMPACTION TEST.

* a. Apparatus. The apparatus consists of the following:

(1) Molds, cylindrical, metal. Molds shall have a detachable base and a collar assembly extending approximately **2-1/2** in. above the top of the mold to retail soil during preparation of compacted specimens of the desired height and volume. Molds having a slight taper to facilitate removal of the specimen after the compaction test are satisfactory provided the taper *

* is no greater than 0.200 in. in diameter per foot of mold height. Capacities and dimensions of the molds shall be as follows:

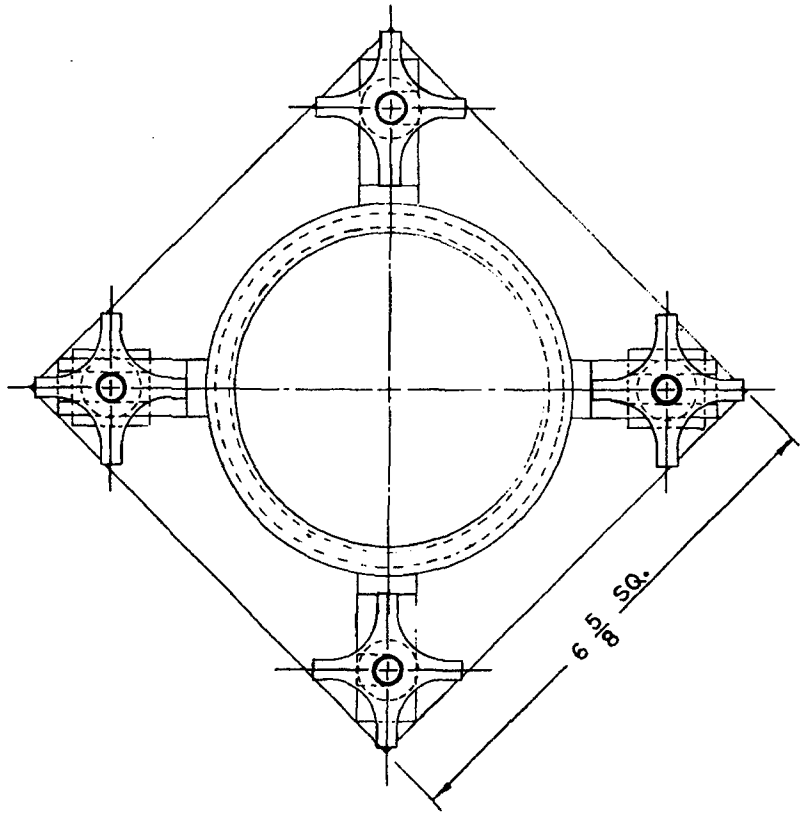
(a) Mold with an average inside diameter of 4.0 ± 0.016 in. and a capacity of 1/30 ± 0.0004 cu ft. Details of a typical mold are shown in Figure 1.

(b) Mold with an average inside diameter of 6.0 ± 0.016 in. and a capacity of 3/40 ± 0.0009 cu ft. The 6.0-in. mold may be similar in construction to that shown in Figure 1, and shall be used for compacting samples containing material that would be retained on the No. 4 sieve but passing the 3/4-in. sieve.

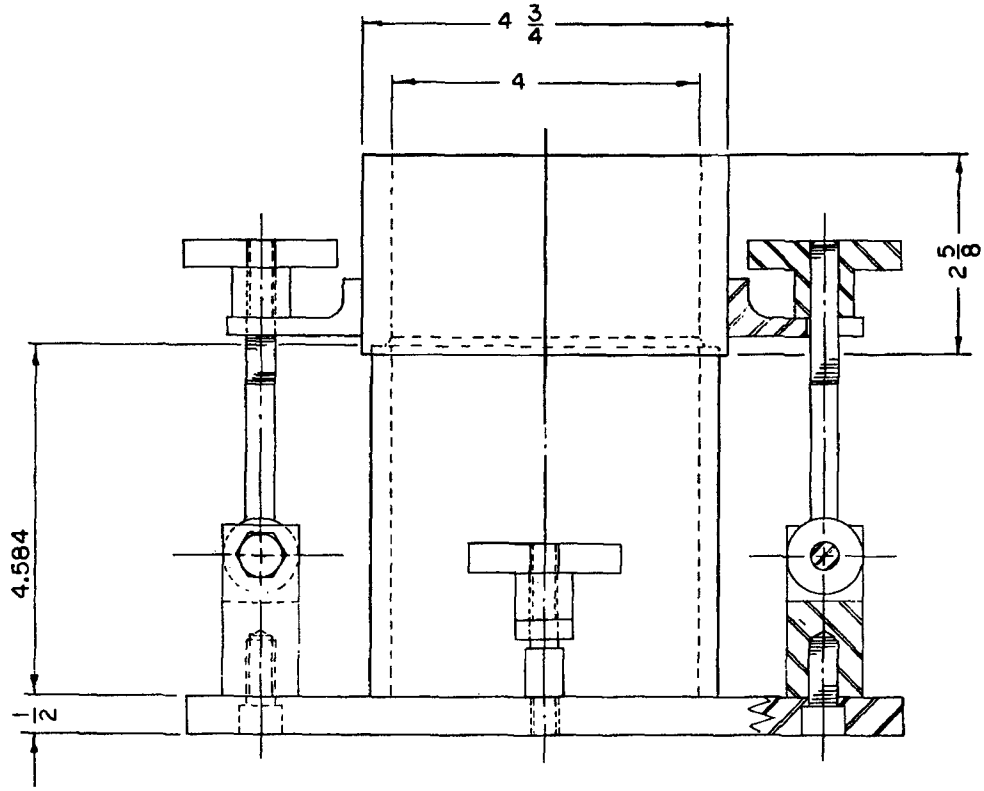
(c) The exact volume of molds should be determined before use and periodically thereafter, and this measured volume is used in calculations.

(2) Rammer, manually or mechanically operated. The rammer shall consist of a drop weight which can be released to fall freely and strike the soil surface. The height of drop shall be controlled so that the weight falls from a height of 12 ± 1/16 above the surface of the soil. The mass of the free falling part of the rammer shall be 5.5 ± 0.02 lb and the striking face of the rammer shall be flat. Rammers must also meet the following requirements:

(a) Manual rammer. The striking face shall be circular with a diameter of 2.0 ± 0.005 in. The rammer shall be equipped with a guide sleeve having sufficient clearance so that the free fall of the rammer shaft and head will not be restricted. The guidesleeve shall have at least four vent holes at each end (eight holes total) located with centers 3/4 ± 1/16 in. from each *



PLAN



ELEVATION

Figure 1. **4.0-in.** diameter compaction mold

* end and space 90 deg apart. The minimum diameter of the vent holes shall be 3/8 in. Additional vent holes or slots may be incorporated in the guidesleeve if desired. Figure 2 illustrates a typical manual rammer.

(b) Mechanical rammer. A mechanical rammer must operate in such a manner as to provide uniform and complete coverage of the specimen surface. The clearance between the rammer and the inside surface of the mold at its smallest diameter shall be 0.10 ± 0.03 in. When used with the 4-in. mold, the specimen contact face shall be circular with a diameter of 2.000 ± 0.005 in. When used with the 6.0-in. mold, the specimen contact face shall be either circular or sector shaped?; if sector shaped, it shall have a radius of 2.90 ± 0.02 in. The sector face rammer shall operate in such a manner that the vertex of the sector is positioned at the center of the specimen.

(c) Calibration of mechanical rammer compactors. The mechanical rammer compactor must be calibrated periodically against the results obtained with the manual rammer. The compactor must be calibrated for the circular foot and, if used, the sector foot. The mechanical compactor shall be calibrate-d before initial use, near the end of each period during which the mold was filled 500 times before use after anything including repairs that may affect test results whenever test results are questionable, and before use after any 6-month period during which the rammer was not calibrated. Procedures for calibrating mechanical compactors are given in Engineer Manual EM 1110-2-1909, Calibration of Laboratory Soils Testing Equipment. *

† The mechanical rammer equipped with a sector shaped foot should not be used for compacting specimens for the California Bearing Ratio (CBR) test described in MIL-STD-621A as CBR values may differ substantially from those obtained on specimens compacted with a rammer having a circular foot.

* (3) Balance having a readability of 1 g, an accuracy of 2 g, and having a capacity sufficient for weighing compacted samples.

(4) Oven (see Appendix I, WATER CONTENT - GENERAL).

(5) Sieves, US Standard 3/4-in. and No. 4 (0.187 in.) conforming to ASTM Designation: E 11, Standard Specification for Wire-Cloth Sieves for Testing Purposes. Large sieves are generally more suitable for this purpose.

(6) Straightedge, steel, at least 1/8 x 1-3/8 x 10 in. and having a beveled edge.

(7) Mixing tools, such as mixing pan, spoon, trowel, spatula, etc. A suitable mechanical device may be used for mixing fine-grained soils with water.

(8) Specimen containers. Seamless metal containers with lids are recommended. The containers should be of a metal resistant to corrosion such as aluminum or stainless steel. Containers 2 in. high by 3-1/2 in. in diameter are adequate.



Figure 2. Manual rammer for standard compaction test

* ('9) Sample splitter or riffle for dividing the samples.

(10) Glass jars, metal cans, or plastic buckets with airtight lids in which to store and cure soil prepared for compaction.

(11) Equipment for determining water contents (see Appendix I, WATER CONTENT - GENERAL).

b. Preparation of Sample. The amount of soil required for the standard compaction test varies with the kind and gradation of the soil to be tested. For soils passing the No. 4 sieve that are to be tested in the 4.0-in. mold, 20 lb of soil is normally sufficient for the test. For samples containing gravel that are to be tested in the 6.0-in. mold, approximately 75 lb of processed material is required. Ordinarily, the soil to be tested shall be air-dried, or dried by means of drying apparatus provided the apparatus will not raise the temperature of the sample above 60° C (140° F). The requirement for fully air-drying soils in preparation for compaction is intended to facilitate soil processing and reduce variability in testing procedures. However in some construction control operations, it may not be practical to completely air dry, rewet, and cure the soil in preparation for compaction. In these instances, the soil is air-dried to some water content near the driest point on the compaction curve and water for preparation of individual test specimens added as needed to obtain the desired range of water contents. Partial air drying of some soils during preparation may lead to compaction results different from those which would be obtained if the soil had been completely air-dried during preparation*. If a procedure other than the standard (fully air-dry, rewet, and cure) procedure is used, comparison tests must be performed for each of *

* the soil types encountered at a given project to verify that there is no differences in results. If differences in results do appear, a procedure that reflects the actual field conditions must be adopted for both design and construction control testing.

Aggregations present in the sample shall be thoroughly broken, but care should be taken that the natural size of the individual particles is not reduced. The material shall then be screened through a **3/4-in.** and a No. 4 sieve. For some soils, it may be desirable to reduce aggregations before the sample is dried. If all the material passes the No. 4 sieve, the sample shall be mixed thoroughly and a representative sample taken to determine the initial water content (see Appendix I, WATER CONTENT - GENERAL). The sample shall then be stored in an air-tight container until ready for processing at different water contents for compaction in the **4.0-in.** mold.

If all the sample passes the **3/4-in.** sieve and contains 5 percent or less material larger than the No. 4 sieve, the plus No. 4 fraction shall be discarded and the test performed using the **4.0-in.** compaction mold. If all the sample passes the **3/4-in.** sieve but contains more than 5 percent material retained on the No. 4 sieve, it shall be tested in the **6-in.** mold. The sample shall be mixed thoroughly after which its initial water content shall be determined. The sample shall then be stored in an airtight container until ready for processing at different water contents for compaction.

If the sample contains some material retained on the **3/4-in.** sieve, but the amount is 5 percent or less, the plus **3/4-in.** fraction shall be removed and discarded and the sample tested in the **6-in.** mold. The initial water content of the sample shall be determined and the sample stored in an airtight *

* container until ready for processing at different water contents for compaction.

If the sample contains more than 5 percent material retained on the 3/4-in. sieve, the test should be performed using the 12-in. compaction mold, the procedures for which are given in Appendix VIA: COMPACTION TEST FOR EARTH-ROCK MIXTURES.

c. Procedure.

(1) Material finer than No. 4 sieve. The procedure for soils finer than the No. 4 sieve shall consist of the following:

(a) Record all identifying information for the sample such as project name or number, boring number, and other pertinent data on a data sheet (see Plate VI-1 for suggested form). Record the compactive effort to be used, size of mold, and initial water content of processed sample.

(b) From the previously prepared sample, weight a quantity of air-dry soil equivalent to 2,500 g oven-dry weight (see paragraph 2d(1)). Thoroughly mix the material with a measured quantity of water sufficient to produce a water content 4 to 6 percentage points below estimated optimum water content. At this water, nonplastic soils tightly squeezed in the palm of the hand will form a cast which will withstand only slight pressure applied by the thumb and fingertips without crumbling; plastic soils will ball noticeably. Store the soil in an airtight container for a sufficient length of time to permit it to absorb the moisture. The time required for complete absorption will vary depending on the type of soil. For nonplastic soils in which

* moisture is readily absorbed, storage is not necessary. For most other soils a minimum curing time of 16 hr is usually adequate.

(c) Repeat step (b) for at least four additional specimens. Increase the water content for each specimen by approximately 2 percentage points over that of the previous specimen.

(d) Weigh the 4.0-in. compaction mold to the nearest gram, and record the weight on the data sheet.

(e) Attach the mold, with collar, to the base plate and place the mold on a uniform, rigid foundation, such as a block or cylinder of concrete weighing not less than 200 lb.

(f) Place an amount of the previously prepared sample in the 4.0-in. mold such that when three such layers have been compacted in the mold, the total compacted height is between 4-518 in. and 5 in.† Compact each layer by 25 uniformly distributed blows from the rammer, with the drop weight falling freely from a height of 12.0 in. In operating the manual rammer, take care to hold the rammer vertical and avoid rebounding the rammer drop weight from the top of the guidesleeve. Apply the blows at a uniform rate not exceeding 1.4 sec per blow. The compaction procedure is illustrated in Figure 3.

(g) Remove the extension collar from the mold. Remove the exposed compacted soil with a knife and carefully trim *

† It is important that the compacted soil just fill the mold with little excess to be struck off. As the amount of material to be struck off varies, the mass of soil to which a constant amount of energy is supplied varies. When the amount of material to be struck off is more than about 1/4 in., the test results become less accurate.



Figure 3. Compacting soil specimen.

the surface even with the top of the mold by means of a straight-edge. Any cavities formed by large particles being pulled out should be carefully patched with material from the trimmings.

(h) Remove the mold with the compacted specimen therein from the base plate, weigh the mold plus wet soil to the nearest gram, and record the weight on the data sheet. When cohesionless soils are being tested there is a possibility of losing the sample if the base plate is removed. For these soils, weigh the entire unit.

* **(i)** Remove the compacted specimen from the mold, and slice it vertically through the center. Take a representative specimen of the material from each of the two parts and determine the water content of each (see Appendix I, WATER CONTENT - GENERAL). The water content specimens shall weigh not less than 100 g. Alternatively, the entire compaction specimen may be used for the water content determination. In this case, the wet weight of specimen for use in computing water content should be redetermined after the specimen is extruded from the compaction mold as some loss of material may occur during transfer of the specimen._

(j) Repeat steps **(d)** through **(i)** for remaining specimens. Compact a sufficient number of test specimens over a range of water contents to establish definitely the optimum water content and maximum density. Generally, five compacted specimens prepared according to the above-described procedure should completely define a compaction curve. However, sometimes more specimens are necessary. To determine if the optimum water content has been reached, compare the wet weights of the various compacted specimens. The optimum water content and maximum density have been reached if the wettest specimens compacted indicate a decrease in weight in relation to drier specimens.

(2) Materials larger than **3/4** in. sieve. The procedure for determining the density and optimum water content of soils containing material retained on the **3/4** in. sieve is the same as that for the finer than **3/4** in. sieve material, except that the test is performed in the **6.0-in.-diam** mold and the number of blows of the compaction rammer is 56 per soil layer instead of 25. This results in equal compactive efforts for the two molds. It is advisable to use the entire compacted specimen for the water content determination. The quantity of soil

* required for each compacted sample will be equivalent to about 5,500 g of oven-dry material.

d. Computations.

(1) Preparation of specimen. The required weight of soil, W'_o, in grams necessary to produce 2,500 g of oven-dry soil is computed as follows:

$$W'_o = W'_s \left(1 + \frac{w_o}{100}\right)$$

where

w_o = initial water content of material (after air-drying)

W'_s = desired weight of oven-dry soil = 2,500 g

The amount of water, W_w, in cc, to be added to the weight of soil, W'_o, to produce specimens at the desired test water contents is computed as follows:

$$W_w = \frac{W'_s (w' - w_o)}{100}$$

where

w' = desired test water content

(2) Quantities obtained in compaction test. The following quantities are obtained for each specimen in the compaction test: *

(a) Weight of compaction mold plus wet soil. The weight of the compaction mold is subtracted from this value to obtain the weight of the soil, W .

(b) The inside volume of the compaction mold. This volume is equal to the volume, V , of the wet soil specimen.

(c) Weight of water content specimen plus tare before and after oven-drying. The tare weight is subtracted from these values to obtain the weight of wet and dry soils for computing water content.

(3) Water content and density. The water content, w , of each compacted specimen shall be computed in accordance with Appendix I, WATER CONTENT - GENERAL. The weight of oven-dry soil, W_s , of each compacted specimen shall be computed according to the formula:

$$\text{Dry weight of specimen} = \frac{\text{weight of wet soil}}{1 + \frac{\text{water content}}{100}}$$

$$W_s = \frac{W}{1 + \frac{w}{100}}$$

The dry weight of the specimen is obtained directly if the entire compacted specimen is used for the water content determination and no loss of material occurs during removal of the specimen from the mold.

The wet unit weight, γ_m, (optional) and the dry unit weight, γ_d, expressed in pounds per cubic foot, shall be computed by the following formulas: *

* $$\text{Wet unit weight} = \frac{\text{weight in g of wet specimen}}{\text{volume in cc of wet specimen}} \times 62.4$$

$$\gamma_m = \frac{W}{V} \times 62.4$$

$$\text{Dry unit weight} = \frac{\text{weight in g of oven-dry specimen}}{\text{volume in cc of wet specimen}} \times 62.4$$

$$\gamma_d = \frac{W_s}{V} \times 62.4$$

These computations may be simplified by use of a mold constant, C , computed as follows:

$$C = \frac{62.4}{V}$$

so that $\gamma_m = CW$ and $\gamma_d = CW_s$.

e. Presentation of Results.

(1) Compaction curve. The results of the standard compaction test shall be presented in the form of a compaction curve on an arithmetic plot as shown in Plate VI-2. The dry densities in pounds per cubic foot are plotted as ordinates and the corresponding water contents in percentage of dry weight as abscissas. The plotted points shall be connected with a smooth curve; for most soils the curve produced is generally parabolic in form. A typical compaction curve is shown in Figure 4. The water content corresponding to the peak of the compaction curve is the optimum water content, and this value shall be recorded to the nearest 0.1 percent. The dry unit weight of the soil in pounds per cubic foot at the optimum water content is the maximum *

* dry density, and this value shall be recorded to the nearest 0.1 lb per cu ft.

(2) Air voids curves. The zero air voids curve (see example in Figure 4) represents the dry density and water content of a soil completely saturated with water. The zero air voids and 90 percent saturation curves shall be shown with the compaction curve in Plate VI-2. Data for plotting these curves for soils with different specific gravities are given in Table VI-1. The specific gravity of the soil used in the compaction test shall be determined in Appendix IV, SPECIFIC GRAVITY.

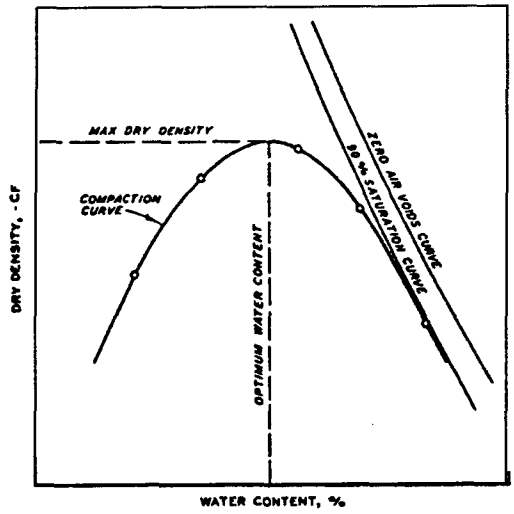


Figure 4. Determination of maximum density and optimum water content

3. MODIFIED COMPACTION TEST. The modified compaction test differs from the standard test in that a greater compactive effort is used which results in higher maximum densities and lower optimum water contents. The apparatus, preparation of sample, and procedure are the same as those used in the standard compaction test, with the following modifications:

a. **Apparatus.** The rammer shall consist of a 10.00-lb weight with an 18.0-in. free drop. If a mechanical rammer is used in performing these tests, the rammer must be calibrated separately for this test in accordance with procedures given in Engineer Manual EM 1110-2-1909, Calibration of Laboratory Soils Testing Equipment. *

* b. Procedure. The soil shall be compacted in five layers of equal thickness. The number of blows per layer shall be the same as for the standard compaction test: 25 blows per layer in the **4.0-in.-diameter** mold, and 56 blows per layer in the **6.0-in.-** diameter mold. The computations and presentation of results shall be the same as those used in the standard compaction test.

4. **15-BLOW** COMPACTION TEST. The **15-blow** compaction test differs from the standard compaction test in that a lesser compactive effort is used resulting in lower maximum densities and higher optimum water contents. The apparatus, preparation of samples, and procedures shall be the same as those used in the standard compaction test **(5.50-lb** weight with a **12.0-in.** free drop) with the following modifications:

a. The **6-in.** mold shall not be used.

b. The number of blows per layer shall be 15.

The computations and presentation of results shall be the same as those used in the standard compaction test.

5. POSSIBLE ERRORS. Following are possible errors that would cause inaccurate determinations of compaction curves for any compactive effort:

a. Aggregations of dried soil not completely broken.

b. Water not thoroughly absorbed into dried soil. Consistent results cannot be obtained unless the soil and water are complete mixed and sufficient time allowed for the soil to absorb the water uniformly. *

* c. Soil reused. Since some soils are affected by recompaction, fresh material must be used for each specimen. Recompaction tends to increase the maximum dry unit weight of some clays and, therefore, decrease the apparent optimum water content.

d. Insufficient number of range of water contents to define compaction curve accurately. See paragraph 2c(1)(j).

e. Improper foundation for compaction mold.

f. Incorrect volume of compaction mold used. The exact inside volume of each mold must be determined before being used.

g. Mechanical compactor not properly calibrated.

h. Human factors in the operation of hand rammer. Variations in results can be caused by not bringing the drop weight to a complete stop before releasing it to fall and compact the soil. If raising and releasing the rammer's drop weight is done too quickly, the drop weight will not be brought to rest before release. If the rammer is not held vertical during operation, the compactive effort will be reduced. The tendency to press the sleeve of the manual rammer into the soil specimen, the way the blows are distributed over the surface of the specimen, and other individual operator characteristics all tend to affect compaction results. By proper instruction and supervision, uniform technique can be maintained within a laboratory; however, it is preferable that all specimens of a given test be compacted by the same person with the same rammer in one sitting.

i. Excessive variation in total depth of compacted specimen. The extension of the specimen into the collar of the mold *

* should not exceed about 1/4 in., and care should be taken that each layer is nearly equal in weight.

j. Water content determination not representative of specimen. This error can be avoided by using the entire specimen for the water content determination.

Table VI-1

Data for Zero Air Voids Curve

Specific Gravity of Soil	Water Content, w, in Per Cent of Dry Weight for Dry Unit Weight, γ_d, in pounds per cubic foot of																			
	50	55	60	65	70	75	80	85	90	95	100	105	110	115	120	125	130	135	140	145
2.40	83.2	71.8	62.4	54.4	47.5	41.6	36.4	31.8	27.7	24.1	20.8	17.8	15.1	12.6	10.4	8.3	6.4	4.6	2.9	1.4
2.45	84.0	72.7	63.2	55.2	48.4	42.4	37.2	32.6	28.6	24.9	21.6	18.6	15.9	13.5	11.2	9.1	7.2	5.4	3.8	2.2
2.50	84.9	73.5	64.1	56.1	49.2	43.2	38.0	33.5	29.4	25.7	22.4	19.5	16.8	14.3	12.0	9.9	8.0	6.2	4.6	3.1
2.51	85.0	73.7	64.2	56.2	49.4	43.4	38.2	33.6	29.5	25.9	22.6	19.6	16.9	14.5	12.2	10.1	8.2	6.4	4.8	3.2
2.52	85.2	73.8	64.4	56.4	49.5	43.6	38.4	33.8	29.7	26.0	22.8	19.8	17.1	14.6	12.4	10.3	8.3	6.6	4.9	3.4
2.53	85.3	74.0	64.5	56.5	49.7	43.7	38.5	33.9	29.8	26.2	22.9	19.9	17.2	14.8	12.5	10.4	8.5	6.7	5.1	3.5
2.54	85.5	74.1	64.7	56.7	49.8	43.9	38.7	34.1	30.0	26.4	23.1	20.1	17.4	14.9	12.7	10.6	8.7	6.9	5.2	3.7
2.55	85.6	74.3	64.8	56.8	50.0	44.0	38.8	34.2	30.2	26.5	23.2	20.2	17.5	15.1	12.8	10.7	8.8	7.0	5.4	3.8
2.56	85.8	74.5	65.0	57.0	50.1	44.2	39.0	34.4	30.3	26.7	23.4	20.4	17.7	15.2	13.0	10.9	9.0	7.2	5.5	4.0
2.57	86.0	74.6	65.1	57.1	50.3	44.3	39.1	34.5	30.5	26.8	23.5	20.6	17.8	15.4	13.1	11.0	9.1	7.3	5.7	4.2
2.58	86.1	74.8	65.3	57.3	50.4	44.5	39.3	34.7	30.6	27.0	23.7	20.7	18.0	15.5	13.3	11.2	9.3	7.5	5.8	4.3
2.59	86.3	74.9	65.4	57.4	50.6	44.6	39.4	34.8	30.8	27.1	23.8	20.9	18.1	15.7	13.4	11.3	9.4	7.6	6.0	4.5
2.60	86.4	75.1	65.6	57.6	50.7	44.8	39.6	35.0	30.9	27.3	24.0	21.0	18.3	15.8	13.6	11.5	9.6	7.8	6.1	4.6
2.61	86.6	75.2	65.7	57.7	50.9	44.9	39.7	35.1	31.1	27.4	24.1	21.2	18.4	16.0	13.7	11.6	9.7	7.9	6.3	4.8
2.62	86.7	75.3	65.9	57.9	51.0	45.1	39.9	35.3	31.2	27.6	24.3	21.3	18.6	16.1	13.9	11.8	9.9	8.1	6.4	4.9
2.63	86.8	75.5	66.0	58.0	51.2	45.2	40.0	35.4	31.4	27.7	24.4	21.4	18.7	16.3	14.0	11.9	10.0	8.2	6.6	5.0
2.64	87.0	75.6	66.2	58.2	51.3	45.4	40.2	35.6	31.5	27.8	24.6	21.6	18.9	16.4	14.2	12.1	10.1	8.4	6.7	5.2
2.65	87.1	75.8	66.3	58.3	51.5	45.5	40.3	35.7	31.6	28.0	24.7	21.7	19.0	16.6	14.3	12.2	10.3	8.5	6.9	5.3
2.66	87.3	75.9	66.5	58.5	51.6	45.7	40.5	35.9	31.8	28.1	24.8	21.9	19.2	16.7	14.4	12.4	10.4	8.7	7.0	5.5
2.67	87.4	76.1	66.6	58.6	51.7	45.8	40.6	36.0	31.9	28.3	25.0	22.0	19.3	16.8	14.6	12.5	10.6	8.8	7.1	5.6
2.68	87.6	76.2	66.7	58.7	51.9	45.9	40.7	36.1	32.1	28.4	25.1	22.2	19.4	17.0	14.7	12.6	10.7	8.9	7.3	5.8
2.69	87.7	76.3	66.9	58.9	52.0	46.1	40.9	36.3	32.2	28.6	25.3	22.3	19.6	17.1	14.9	12.8	10.9	9.1	7.4	5.9
2.70	87.8	76.5	67.0	59.0	52.2	46.2	41.0	36.4	32.3	28.7	25.4	22.4	19.7	17.3	15.0	12.9	11.0	9.2	7.6	6.0
2.71	88.0	76.6	67.2	59.2	52.3	46.3	41.1	36.6	32.5	28.8	25.5	22.6	19.9	17.4	15.1	13.0	11.1	9.3	7.7	6.2
2.72	88.1	76.8	67.3	59.3	52.4	46.5	41.3	36.7	32.6	29.0	25.7	22.7	20.0	17.5	15.3	13.2	11.3	9.5	7.8	6.3
2.73	88.2	76.9	67.4	59.4	52.6	46.6	41.4	36.8	32.7	29.1	25.8	22.8	20.1	17.7	15.4	13.3	11.4	9.6	8.0	6.4
2.74	88.4	77.0	67.6	59.6	52.7	46.7	41.5	37.0	32.9	29.2	25.9	23.0	20.3	17.8	15.5	13.4	11.5	9.7	8.1	6.6
2.75	88.5	77.2	67.7	59.7	52.8	46.9	41.7	37.1	33.0	29.4	26.1	23.1	20.4	17.9	15.7	13.6	11.7	9.9	8.2	6.7
2.76	88.6	77.3	67.8	59.8	53.0	47.0	41.8	37.2	33.1	29.5	26.2	23.2	20.5	18.1	15.8	13.7	11.8	10.0	8.4	6.8
2.77	88.8	77.4	68.0	60.0	53.1	47.1	41.9	37.4	33.3	29.6	26.3	23.4	20.7	18.2	15.9	13.8	11.9	10.1	8.5	7.0
2.78	88.9	77.5	68.1	60.1	53.2	47.3	42.1	37.5	33.4	29.8	26.5	23.5	20.8	18.3	16.1	14.0	12.1	10.3	8.6	7.1
2.79	89.0	77.7	68.2	60.2	53.4	47.4	42.2	37.6	33.5	29.9	26.6	23.6	20.9	18.5	16.2	14.1	12.2	10.4	8.8	7.2
2.80	89.2	77.8	68.3	60.3	53.5	47.5	42.3	37.7	33.7	30.0	26.7	23.8	21.0	18.6	16.3	14.2	12.3	10.5	8.9	7.4
2.81	89.3	77.9	68.5	60.5	53.6	47.7	42.5	37.9	33.8	30.1	26.8	23.9	21.2	18.7	16.4	14.4	12.4	10.7	9.0	7.5
2.82	89.4	78.1	68.6	60.6	53.7	47.8	42.6	38.0	33.9	30.3	27.0	24.0	21.3	18.8	16.6	14.5	12.6	10.8	9.1	7.6
2.83	89.5	78.2	68.7	60.7	53.9	47.9	42.7	38.1	34.0	30.4	27.1	24.1	21.4	19.0	16.7	14.6	12.7	10.9	9.3	7.7
2.84	89.7	78.3	68.8	60.8	54.0	48.0	42.8	38.2	34.2	30.5	27.2	24.3	21.5	19.1	16.8	14.7	12.8	11.0	9.4	7.9
2.85	89.8	78.4	69.0	61.0	54.1	48.2	43.0	38.4	34.3	30.6	27.3	24.4	21.7	19.2	16.9	14.9	12.9	11.2	9.5	8.0
2.86	89.9	78.5	69.1	61.1	54.2	48.3	43.1	38.5	34.4	30.8	27.5	24.5	21.8	19.3	17.1	15.0	13.1	11.3	9.6	8.1
2.87	90.0	78.6	69.2	61.2	54.4	48.4	43.2	38.6	34.5	30.9	27.6	24.6	21.9	19.5	17.2	15.1	13.2	11.4	9.8	8.2
2.88	90.1	78.8	69.3	61.3	54.5	48.5	43.3	38.7	34.7	31.0	27.7	24.7	22.0	19.6	17.3	15.2	13.3	11.5	9.9	8.3
2.89	90.3	78.9	69.5	61.5	54.6	48.6	43.4	38.9	34.8	31.1	27.8	24.9	22.2	19.7	17.4	15.3	13.4	11.6	10.0	8.5
2.90	90.4	79.0	69.6	61.6	54.7	48.8	43.6	39.0	34.9	31.2	28.0	25.0	22.3	19.8	17.6	15.5	13.5	11.8	10.1	8.6
2.95	91.0	79.6	70.2	62.2	55.3	49.3	44.1	39.6	35.5	31.8	28.5	25.6	22.9	20.4	18.1	16.0	14.1	12.3	10.7	9.2
3.00	91.5	80.2	70.7	62.7	55.9	49.9	44.7	40.1	36.0	32.4	29.1	26.1	23.4	21.0	18.7	16.6	14.7	12.9	11.3	9.7
3.05	92.1	80.7	71.3	63.3	56.4	50.5	45.3	40.7	36.6	32.9	29.6	26.7	24.0	21.5	19.2	17.2	15.2	13.5	11.8	10.3
3.15	93.1	81.8	72.3	64.3	57.4	51.5	46.3	41.7	37.6	34.0	30.7	27.7	25.0	22.5	20.3	18.2	16.3	14.5	12.8	11.3

Note: Zero air voids curve equivalent to a degree of saturation, S , equal to 100 percent.

$$w = S\left(\frac{\gamma_w}{\gamma_d} - \frac{1}{G_s}\right)$$

where

w = water content, percent

S = degree of saturation, percent

γ_w = unit weight of water, lb per cu ft = 62.43

γ_d = dry unit weight of soil, lb per cu ft

G_s = specific gravity of soil solids

This equation may also be used to determine curves representing degrees of saturation other than 100 percent.

*

COMPACTION TEST

Date _____

Project _____

Boring No. _____ Sample No. _____

Mold No. _____ inch diam mold | Volume of mold, V, in cc = _____

Mold constant, C = 62.4 ÷ V = _____ | Initial water content, w_o = _____

_____ blows per each of _____ layers, with _____ lb rammer _____ inch drop

Specimen No.								
Preparation of specimen								
$100 + w_o$								
Weight in grams: Oven-dry soil	W'_s							
Weight in grams: Wet soil = $\frac{W'_s(100 + w_o)}{100}$	W'_o							
Weight in grams: Tare								
Weight in grams: Tare plus wet soil								
Test water content	w'	%	%	%	%	%	%	
Water added = $\frac{W'_s (w' - w_o)}{100}$ in cc								
Compacted specimen								
Weight in grams: Mold plus wet soil								
Weight in grams: Mold tare								
Weight in grams: Wet soil	W							
Weight in grams: Dry soil = $\frac{100\ W}{100 + w}$	W_s							
Water content = $\frac{W - W_s}{W_s} \times 100$	w	%	%	%	%	%	%	
lb per cu ft: Wet unit wt = CW	γ_m							
lb per cu ft: Dry unit wt = CW_s	γ_d							
Water content determinations								
Specimen No.								
Tare No.								
Weight in grams: Tare plus wet soil								
Weight in grams: Tare plus dry soil								
Weight in grams: Water	W_w							
Weight in grams: Tare								
Weight in grams: Dry soil	W_s							
Water content = $\frac{W_w}{W_s} \times 100$	w	%	%	%	%	%	%	

Remarks _____

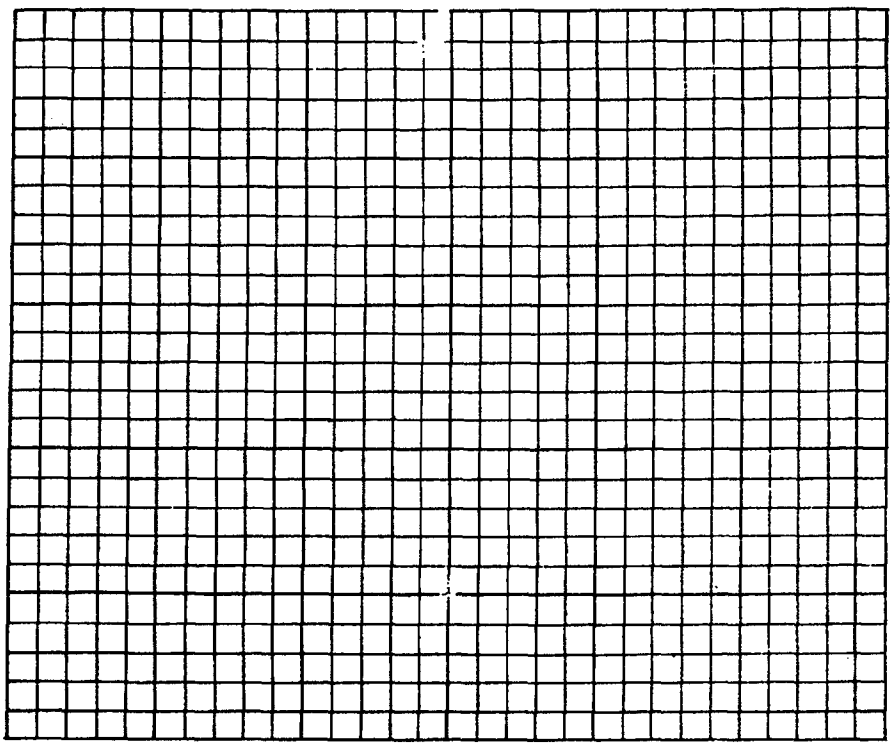
Technician _____ Computed by _____ Checked by _____

ENG FORM 1 JUN 68 3843

D41501 PLATE VI-1

*

*



_____compaction test

_____blows per each of_____layers, with_____lb rammer and

_____inch drop. _____inch diameter mold

Sample No.	Elev or Depth	Classification	G	LL	PL	% > No. 4	% > 3/4 in.

Sample No.				
Natural water content, percent				
Optimum water content, percent				
Max dry density, lb/cu ft				

Remarks	Project
	Area
	Boring No. / Date
	COMPACTION TEST REPORT

ENG FORM 1 JUN 65 2091 PREVIOUS EDITIONS ARE OBSOLETE. (TRANSLUCENT)

*

APPENDIX VIA:

COMPACTION TEST FOR EARTH-ROCK MIXTURES

1. INTRODUCTION. In Appendix VI, procedures are given for the standard effort compaction test using samples having particles finer than the 3/4-in. sieve sizes. The procedures outlined in this appendix are for the standard effort compaction test using material with particles larger * than 3/4-in. and finer than 2-in. sieve sizes. This method should be used for testing material containing particles larger than 3/4-in. * sieve sizes if these particles exceed 10 percent by weight of the total sample. If less than 5 percent by weight of the total sample is finer than the No. 200 sieve, maximum density should be determined by vibratory methods.

The test method outlined is comparable to the standard test in that (a) the compactive effort applied is 12,300 ft-lb/cu ft and (b) the equipment has been devised to maintain ratios between mold diameter, rammer diameter, and maximum particle size of the test specimen similar to those ratios used in the standard test in Appendix VI.

2. APPARATUS. The apparatus shall consist of the following:

a. Cylindrical mold, with an ID of 12.0 in. ± 0.1 in., a height of 12.0 in. ± 0.1 in., and a detachable collar approximately 2-1/2 in. high. The mold and collar assembly should be constructed to fasten to a detachable baseplate. Details of a typical assembly are shown in Figure 1.

b. Hand rammer, metal, of the sliding-weight, fixed-head type with a 4-in.-diameter face and a free-falling weight of ii.50 lb ± 0.05 lb. The rammer should be equipped with a guide such that the height of fall of the sliding weight is 24.00 in. ± 0.05 in. Details of a typical rammer are shown in Figure 2.

c. Balances sensitive to 0.1 lb with a capacity of 250 lb.

d. Oven, forced-draft type, 10- to 12-cu-ft capacity: automatically

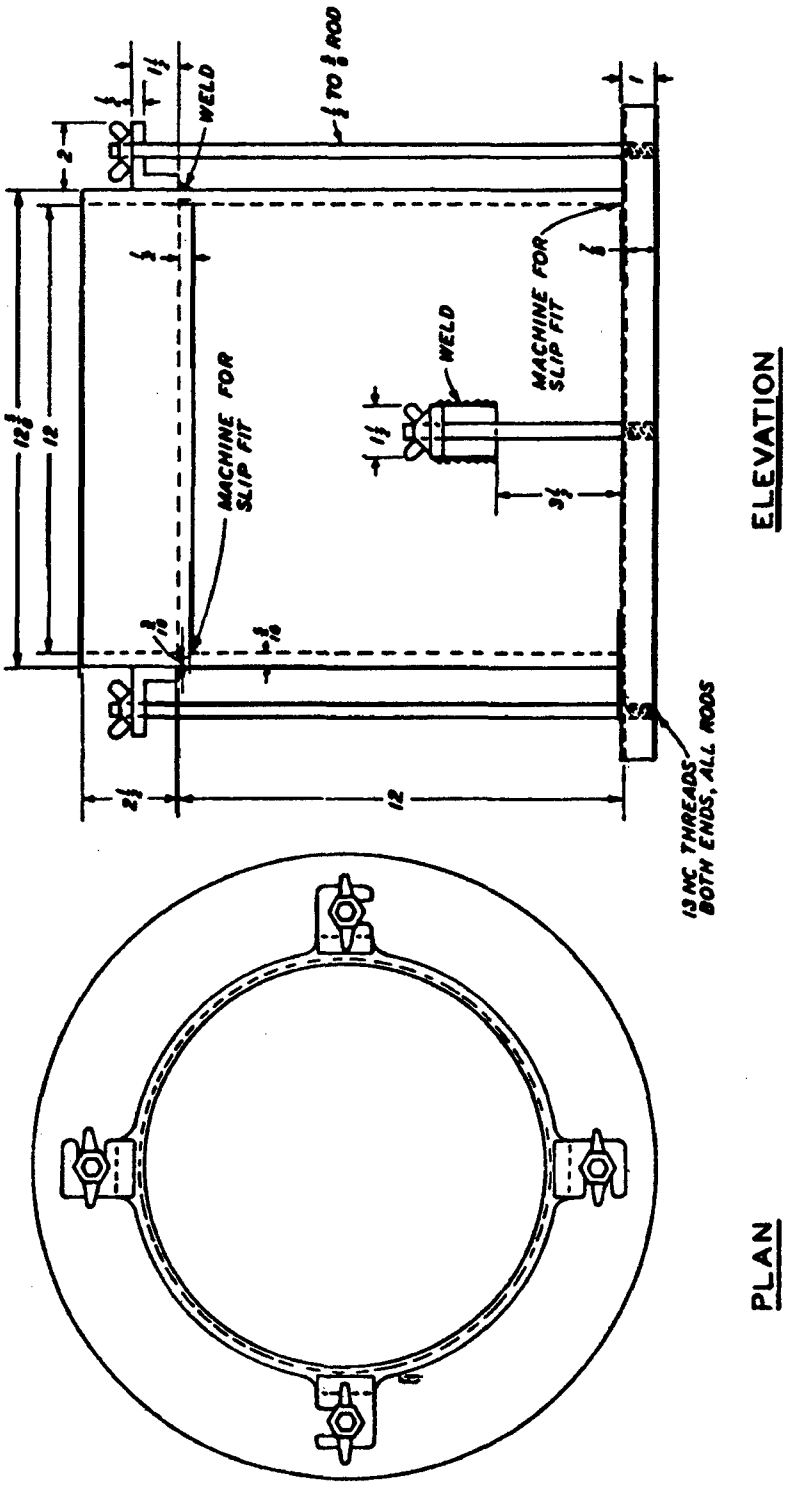


Figure 1. 12-in. compactior ɔld with collar and baseplate

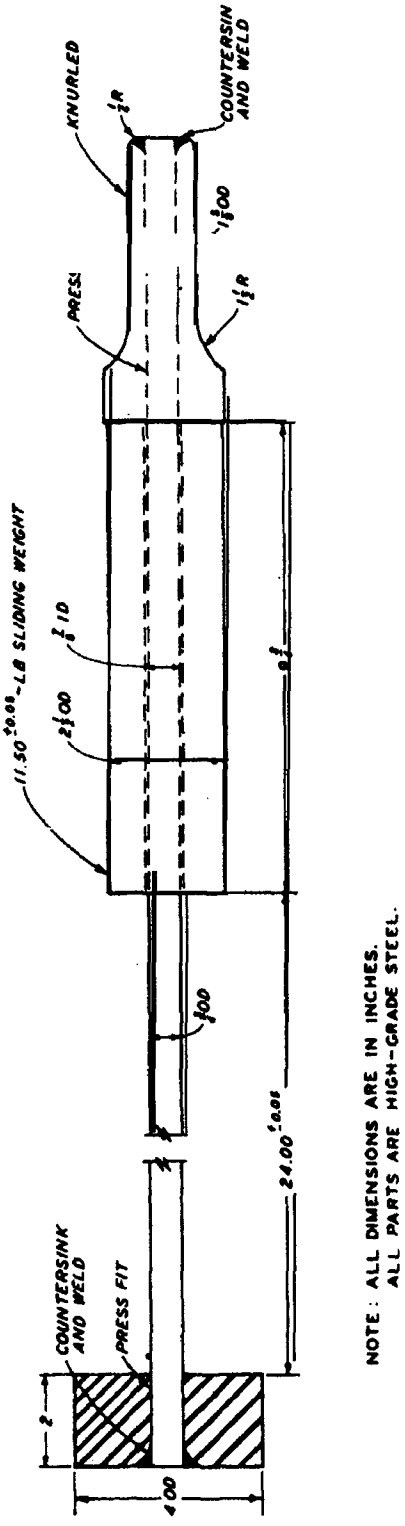


Figure 2. Compaction rammer for 12-in. mold

controlled to maintain a uniform temperature of 110 ± 5 C.

e. Pans, drying, of aluminum or other corrosion-resistant metal, with a capacity of at least 0.5 cu ft. Roasting pans 18 in. by 24 in. by 4 in. are satisfactory.

f. Sieves, U. S. Standard, large diameter type, ranging from 4-in. openings to the No. 4 size, and a mechanical sieve shaker. Sieves with 3/8-, 1/2-, 3/4-, 1-, 1-1/2-, 2-, 3-, and 4-in. openings are normally required.

g. Containers, corrosion resistant, with a capacity of at least 1 cu ft and having airtight lids.

h. Shovel, hand, square-edged, and a mortar box having a capacity of at least 4 cu ft.

i. Straightedge, steel, at least 16 in. long, 3/8 in. thick, and 1 in. wide with a beveled edge.

j. Graduates, hand scoop, trimming knife, wire brush, and rubber-head hammer.

3. QUANTITY OF SAMPLE. At least 700 lb.of sample is required having particles finer than the 2-in. sieve sizes, If the field sample contains quantities of particles larger than the 2-in. sizes, the total sample weight required must be increased to permit removal of oversize particles.

4. PROCESSING OF SAMPLE. a. Record on a work sheet (Plate VIA-l) identifying information for the sample, including visual classification.

b. Spread the material in flat pans and air-dry the entire sample. Other means, such as ovens and heat lamps, may be used to accelerate drying if the maximum drying temperature is 60 C.

c. Reduce all aggregates, or lumps formed during drying, of fine-grained material to particles finer than the No. 4 sieve. With a wire brush or other means, remove all fine-grained material that may be clinging to rock sizes, taking care not to lose the fine-grained material.

d. Separate all the material using a set of sieves ranging from the largest particle size in the sample to the No. 4 sieve. The total sample must be processed to determine the as-received gradation.

e. Place the material retained on each sieve and that passing the No. 4 sieve in separate containers, weigh the contents of each, and compute the percent of the total sample retained on each sieve as follows:

$$\% \text{ Retained} = \frac{\text{dry weight of material retained on sieve}}{\text{dry weight of total sample}} \times 100\%$$

f. If 10 percent or less of a field sample is retained on the 2-in. sieve, the particles larger than this size should be discarded and replacement is not necessary.

g. If more than 10 percent of a field sample is retained on the 2-in. sieve, it will be necessary to remove the plus 2-in. sizes and replace them with an equal weight of material between the 2-in. and No. 4 sieve sizes. The gradation of the replacement material must be the same relative gradation as that of the total sample between the 2-in. and the No. 4 sieve sizes. The percent passing the No. 4 sieve remains constant and is equal to the percent passing the No. 4 sieve for the total as-received sample. For each sieve between the 2-in. and the No. 4 sizes, the percent required to replace the plus 2-in. sizes is computed as follows:

$$\text{Replacement } \% = \text{total } \% \text{ of } +2\text{-in. sizes} \times \frac{\% \text{ retained on one sieve}\dagger}{\text{total } \% \text{ between 2-in. and No. 4 sieve}}$$

For each sieve, add the "Replacement %" to the "% Retained" on that sieve initially. This gives the percent by weight of a test specimen required for each sieve size in order to reconstitute a specimen with the

† Any sieve between 2-in. and No. 4 sieve sizes.

+2-in. sizes replaced with sizes ranging from the 2-in. to the No. 4 sizes. Typical results are tabulated in Plate VIA-1. A typical as-received gradation and test gradation is shown in Plate VIA-2.

5. SPECIAL CONSIDERATIONS. In materials of a heterogeneous nature, such as mixtures of sandstones, siltstones, and shale, the large particles may be siltstone or sandstone, while the smaller size particles may be shale. For materials of this type, when particles larger than the 2-in. sieve sizes are removed for preparation of the test specimen, replacement must be made using the same types of materials "scalped off," or removed. For example, oversize sandstone particles must be removed and replaced, where applicable, with smaller particles of sandstone.

6. PREPARATION OF TEST SPECIMEN. a. Prepare 130 lb of processed air-dried material for the test specimen by combining the weight of material required from each sieve size (refer to typical work sheet, Plate VIA-1).

b. Thoroughly mix the material for the test specimen with a measured quantity of water sufficient to produce a water content 4 or 5 percentage points below the estimated optimum water content of the entire sample. This can be determined only by judgment and experience.

c. Store the moistened sample in an airtight container for a minimum of 16 hr.

d. Prepare material for at least four additional test specimens by repeating steps in paragraphs 6a through 6c. Increase the water content of each specimen by approximately 2 percentage points over that of the previous specimen.

7. COMPACTION PROCEDURE. a. Weigh the compaction mold to the nearest 0.1 lb, determine its inside volume to the nearest 0.001 cu ft, and record the data.

b. Attach the collar to the compactor mold, clamp the mold securely to the baseplate and place the assembly on a level, rigid foundation

made of a concrete cube having a total weight of at least 200 lb. This concrete foundation should not be covered with a rnetal plate.

c. Mix the cured material thoroughly to attain a uniform water distribution.

d. Place a sufficient quantity of the test specimen in the mold to give a compacted layer between 4.0 and 4.5 in. thick. Compact each layer by applying 140 uniformly distributed blows of the rammer, with the 11.5-lb weight falling freely from a height of 24.0 in. compact three equal layers in this manner, taking care to seat the rammer face flush with the soil surface before each blow and to keep the rammer assembly vertical during testing. Use just enough material to finish with less than 1 -in. of sample protruding above the top of mold.

e. Detach the extension collar, taking care not to disturb the soil mass extending above the top of the mold. Trim the surface exactly even with the top of the mold. Fill any cavities formed by removal of particles during trimming with material from the trimrnings and press this filling material firmly into place. Clean excess material from the lip of the mold.

f. Weigh the mold and compacted specimen to the nearest 0.1 lb and record the data.

g. Remove the entire test specimen from the mold, spread it in flat drying pans, and determine its water content. For most specimens, this requires at least 16 hr oven-drying time. Shorter drying times may be used if a constant weight is attained.

h. Repeat the steps in paragraphs 7c through 7g for a sufficient number of specimens over a range of water contents to establish the optimum water content and dry density. Five specimens will usually define the compaction curve accurately. Fresh material, not previously compacted, should be used for all tests.

i. For tests in which degradation of particles due to compaction is significant, determine the after-compaction gradation of at least two total specimens from each test series.

8. COMPUTATIONS. The computations shall consist of the following:

a. Compute the water content of each compacted specimen as follows :

$$\text{Water content, } W, \% = \frac{W_w}{W_s} \times 100$$

where

W_w = wet weight of total specimen minus its oven-dry weight (lb)

W_s = oven-dry weight of specimen (lb)

b. Compute the dry unit weight of each compacted specimen as follows:

$$\text{Dry unit weight, } \gamma, \text{ pcf} = \frac{W_s}{V}$$

where V = volume of the compaction mold (cu ft).

9. PRESENTATION OF RESULTS. Present the results of the test on ENG FORM 2091.

a. Compaction Curve. Plot the dry unit weight, in pounds per cubic foot, as the ordinate and the corresponding water contents, in percentages of dry weight, as the abscissa, on an arithmetic plot. Connect the plotted points with a smooth curve. The water content at the peak of the curve is the optimum water content and the corresponding dry unit weight is the maximum. Record the optimum water content to the nearest 0.1 percent and the maximum dry unit weight to the nearest 0.1 pcf.

b. Zero Air Voids and 90 Percent Saturation Curves. Using the weighted average of the specific gravity of the plus No. 4 and the minus No. 4 material, compute and plot the zero air voids curve and the curve representing the line of 90 percent saturation.

10. POSSIBLE ERRORS. The following errors can cause inaccurate results:

a. Aggregations of air-dried soil not completely reduced to

finer particles during processing.

b. Water not thoroughly absorbed into dried material due to insufficient mixing and curing time.

c. Material reused after compaction.

d. Insufficient number of tests to define compaction curve accurately.

e. Improper foundation for mold during compaction.

f. Incorrect volume or weight of compaction mold.

g. Incorrect rammer weight and height of fall.

*h. Excessive material extending into the extension collar at the end of compaction.

i. Improper or insufficient distribution of blows over the soil surface.

j. Tendency to press the head of the rammer against the specimen before letting the weight fall.

k. Insufficient drying of sample for water content determination.

*To eliminate the need for trimming away the excess material extending into the collar, an alternative may be used. Determine, using the sand-cone method, the volume of the empty collar above the top of the mold and the volume of the collar partially filled with the protruding specimen. The difference is the volume of the compacted excess material. Weigh the entire specimen without trimming the excess and determine the moist unit weight.

PROJECT _____
LAB. N O . _____
DATE _____
TESTED BY _____

COMPACTION TEST
WORK SHEET

TRIAL NO.	WET WT SPECIMEN & MOLD LB	MOLD WEIGHT LB	SPECIMEN WET WT LB	MOISTURE CONTENT		SPECIMEN DRY WT LB	VOLUME OF MOLD (SPECIMEN) FT³	DRY DENSITY LB/CU FT
				WT OF WATER LB	PERCENT MOISTURE			
1	156.3	64.2	92.1	3.7	4.2	88.4	0.0783	112.9
2	163.6	64.2	99.4	5.5	5.9	93.9	0.0783	119.9
3	171.7	64.2	107.5	8.1	8.1	99.4	0.0783	126.9
4	171.8	64.2	107.6	10.0	10.2	97.6	0.0783	124.6
5	168.8	64.2	104.6	11.0	11.8	93.6	0.0783	119.5

GRADATION

SIEVE SIZE	ORIGINAL SAMPLE		TEST SPECIMEN			WT FOR COMPACTION SPECIMEN
	WT RETAINED	% RETAINED	% ORIGINAL	% REPLACEM'T	% TOTAL	
2"	104.0	13.0	0.0	0.0	0.0	0.0
1-1/2"	40.0	5.0	5.0	1.2	6.2	8.1
1"	72.0	9.0	9.0	2.1	11.1	14.4
3/4"	72.0	9.0	9.0	2.1	11.1	14.4
1/2"	80.0	10.0	10.0	2.2	12.2	15.9
3/8"	64.0	8.0	8.0	1.9	9.9	12.9
#4	120.0	15.0	15.0	3.5	18.5	24.0
PAN	248.0	31.0	31.0	0.0	31.0	40.3
TOTAL	800.0	100.0	87.0	13.0	100.0	130.0

REMARKS 140 BLOWS/EACH OF 3 LAYERS WITH 11.5-LB RAMMER AND 24-IN. DROP. DIAMETER O F MOLD = 12 IN. _____

STANDARD ✓

MODIFIED _____

PLATE VIA-I

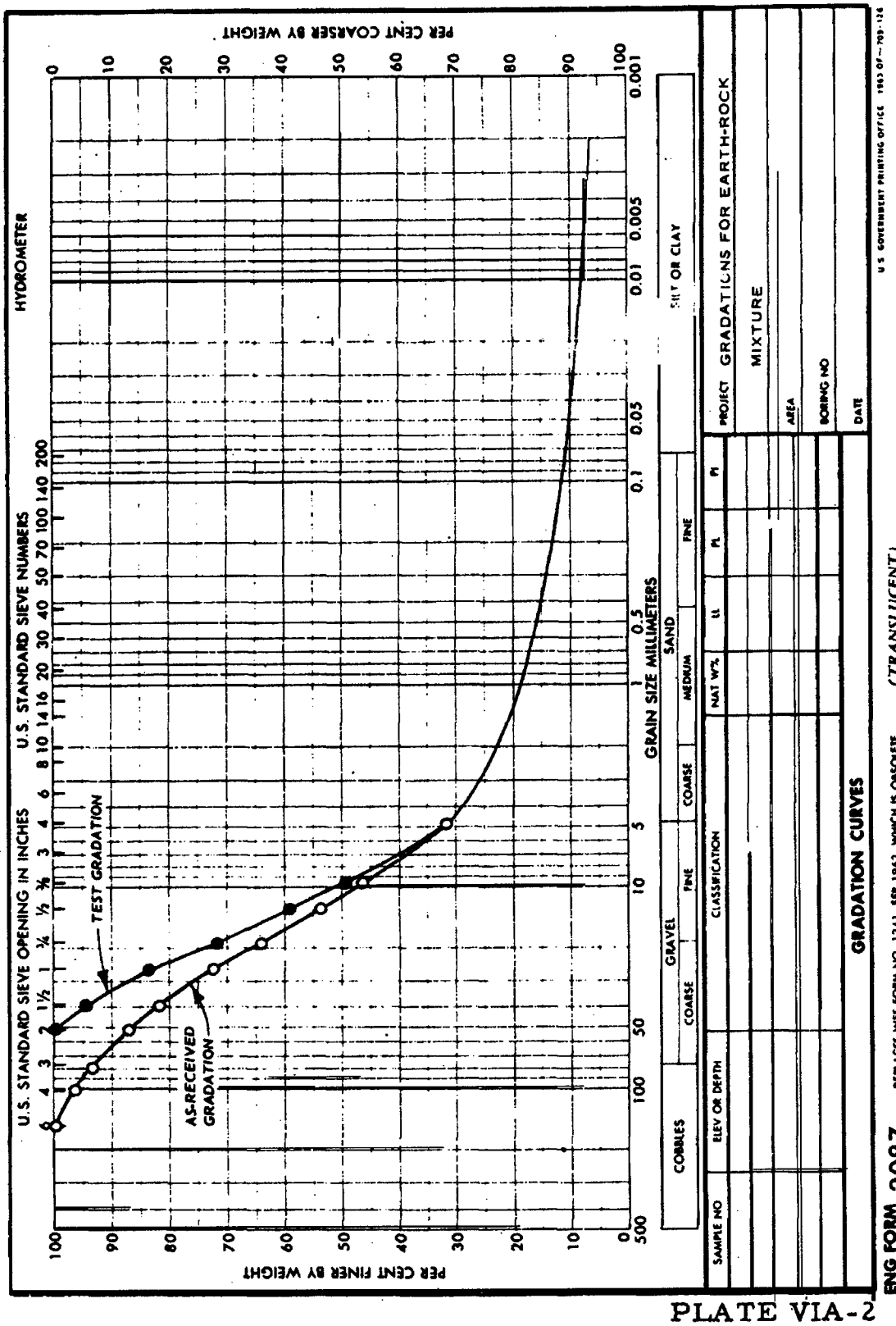

PER CENT COARSER BY WEIGHT
PER CENT FINER BY WEIGHT
HYDROMETER
U.S. STANDARD SIEVE NUMBERS
U.S. STANDARD SIEVE OPENING IN INCHES
GRAIN SIZE MILLIMETERS
TEST GRADATION
AS-RECEIVED GRADATION
COBBLES
GRAVEL
SAND
SILT OR CLAY
COARSE
FINE
MEDIUM
PROJECT GRADATIONS FOR EARTH-ROCK MIXTURE
AREA
BORING NO
DATE
SAMPLE NO
ELEV OR DEPTH
CLASSIFICATION
NAT W%
LL
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PI
GRADATION CURVES
(TRANSLUCENT)
ENG FORM 2087
1 MAY 63
REPLACES WES FORM NO 1241, SEP 1962, WHICH IS OBSOLETE.

PLATE VIA-2

APPENDIX VII:

PERMEABILITY TESTS

1. **DARCY'S LAW FOR FLOW OF WATER THROUGH SOIL.** The flow of water through a soil medium is assumed to follow Darcy's law:

$$q = k\,i\,A$$

where q = rate of discharge through a soil of cross-sectional area A

k = coefficient of permeability

i = hydraulic gradient: the loss of hydraulic head per unit distance of flow

The application of Darcy's law to a specimen of soil in the laboratory is illustrated in Figure 1. The coefficient of permeability, k (often termed

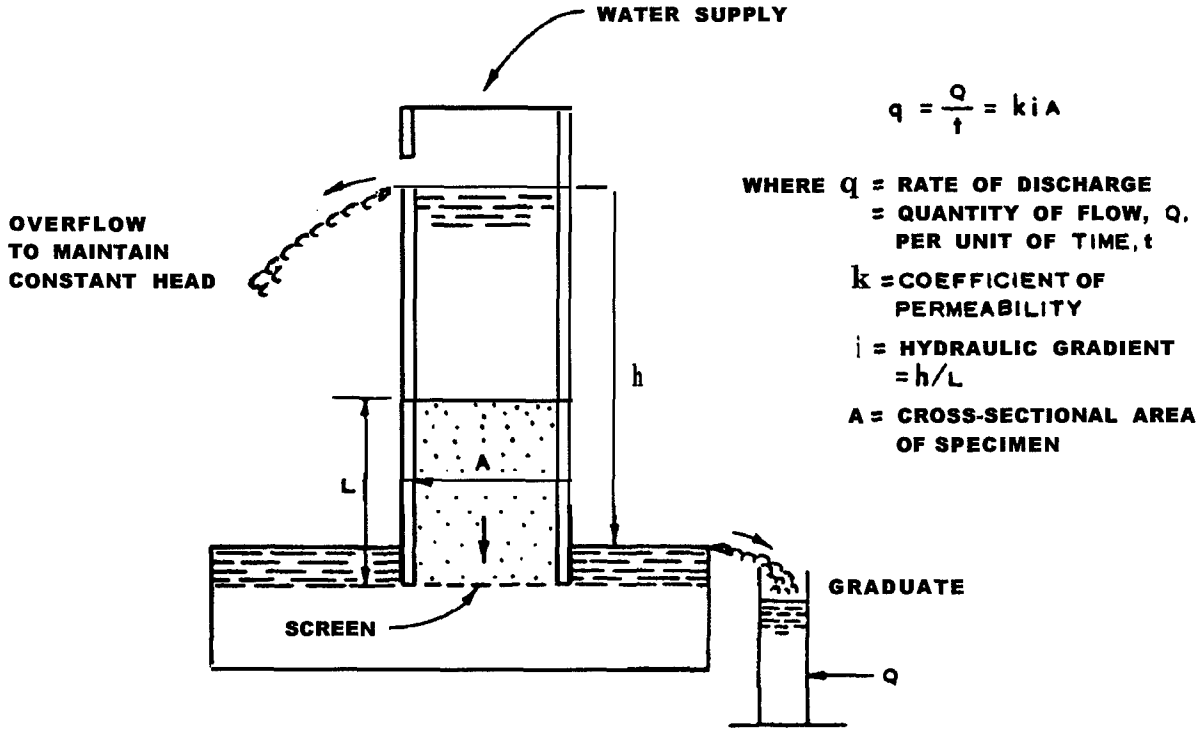


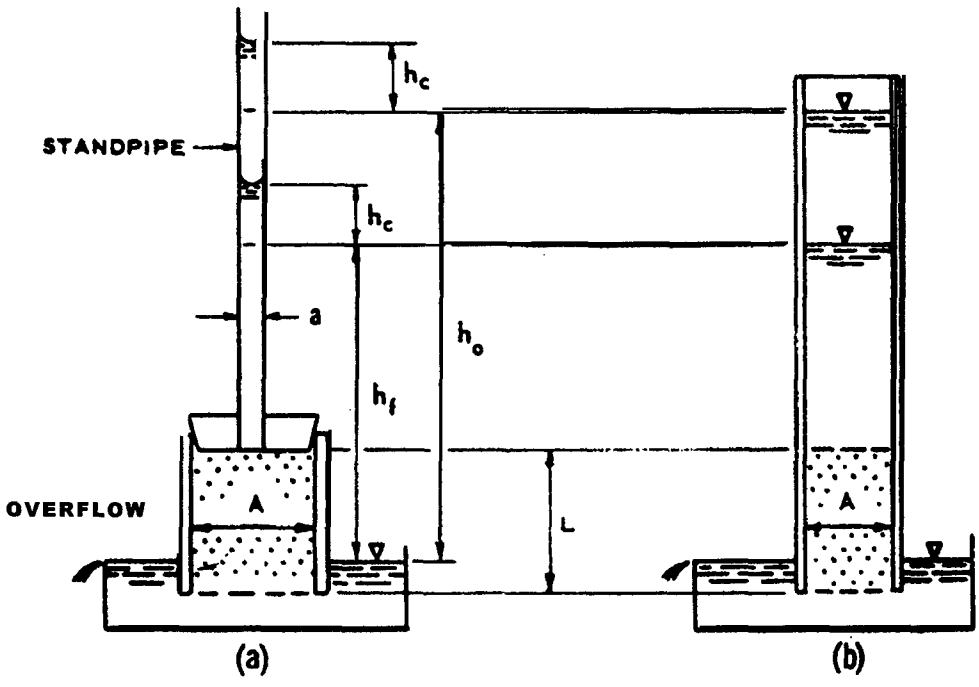
Figure 1. Flow of water through soil

"permeability"), is defined as the rate of discharge of water at a temperature of 20 C under conditions of laminar flow through a unit cross-sectional area of a soil medium under a unit hydraulic gradient. The coefficient of permeability has the dimensions of a velocity and is usually expressed in centimeters per second. The permeability of a soil depends primarily on the size and shape of the soil grains, the void ratio of the soil, the shape and arrangement of the voids, and the degree of saturation.

Permeability computed on the basis of Darcy's law is limited to the conditions of laminar flow and complete saturation of the voids. In turbulent flow, the flow is no longer proportional to the first power of the hydraulic gradient. Under conditions of incomplete saturation, the flow is in a transient state and is time-dependent. The laboratory procedures presented herein for determining the coefficient of permeability are based on the Darcy conditions of flow. Unless otherwise required, the coefficient of permeability shall be determined for a condition of complete saturation of the specimen. Departure from the Darcy flow conditions to simulate natural conditions is sometimes necessary; however, the effects of turbulent flow and incomplete saturation on the permeability should be recognized and taken into consideration.

2. TYPES OF TESTS AND EQUIPMENT. a. Types of Tests. (1) Constant-head test. The simplest of all methods for determining the coefficient of permeability is the constant-head type of test illustrated in Figure 1. This test is performed by measuring the quantity of water, Q, flowing through the soil specimen, the length of the soil specimen, L, the head of water, h, and the elapsed time, t. The head of water is kept constant throughout the test. For fine-grained soils, Q is small and may be difficult to measure accurately. Therefore, the constant-head test is used principally for coarse-grained soils (clean sands and gravels) with k values greater than about 10×10^{-4} cm per sec.

(2) Falling-head test. The principle of the falling-head test is illustrated in Figure 2. This test is conducted in the same manner as



USING SETUP SHOWN IN (a), THE COEFFICIENT OF PERMEABILITY IS DETERMINED AS FOLLOWS:

$$k = \frac{La}{At} \ln \frac{h_o}{h_f} = 2.303 \frac{La}{At} \log_{10} \frac{h_o}{h_f}$$

USING SETUP SHOWN IN (b), THE COEFFICIENT OF PERMEABILITY IS DETERMINED AS FOLLOWS:

$$k = \frac{L}{t} \ln \frac{h_o}{h_f} = 2.303 \frac{L}{t} \log_{10} \frac{h_o}{h_f}$$

WHERE: h_c = HEIGHT OF CAPILLARY RISE

a = INSIDE AREA OF STANDPIPE

A = CROSS-SECTIONAL AREA OF SPECIMEN

L = LENGTH OF SPECIMEN

h_o = HEIGHT OF WATER IN STANDPIPE ABOVE DISCHARGE LEVEL MINUS h_c AT TIME, t_o

h_f = HEIGHT OF WATER IN STANDPIPE ABOVE DISCHARGE LEVEL MINUS h_c AT TIME, t_f

t = ELAPSED TIME, $t_f - t_o$

Figure 2. Principle of falling-head test

the constant-head test, except that the head of water is not maintained constant but is permitted to fall within the upper part of the specimen container or in a standpipe directly connected to the specimen. The quantity of water flowing through the specimen is determined indirectly by computation. The falling-head test is generally used for less pervicus soils (fine sands to fat clays) with k values less than 10×10^{-4} cm per sec.

b. Equipment. The apparatus used for permeability testing may vary considerably in detail depending primarily on the condition and character of the sample to be tested. Whether the sample is fine-grained or coarse-grained, undisturbed, remolded, or compacted, saturated or nonsaturated will influence the type of apparatus to be employed. The basic types of apparatus, grouped according to the type of specimen container (permeameter), are as follows:

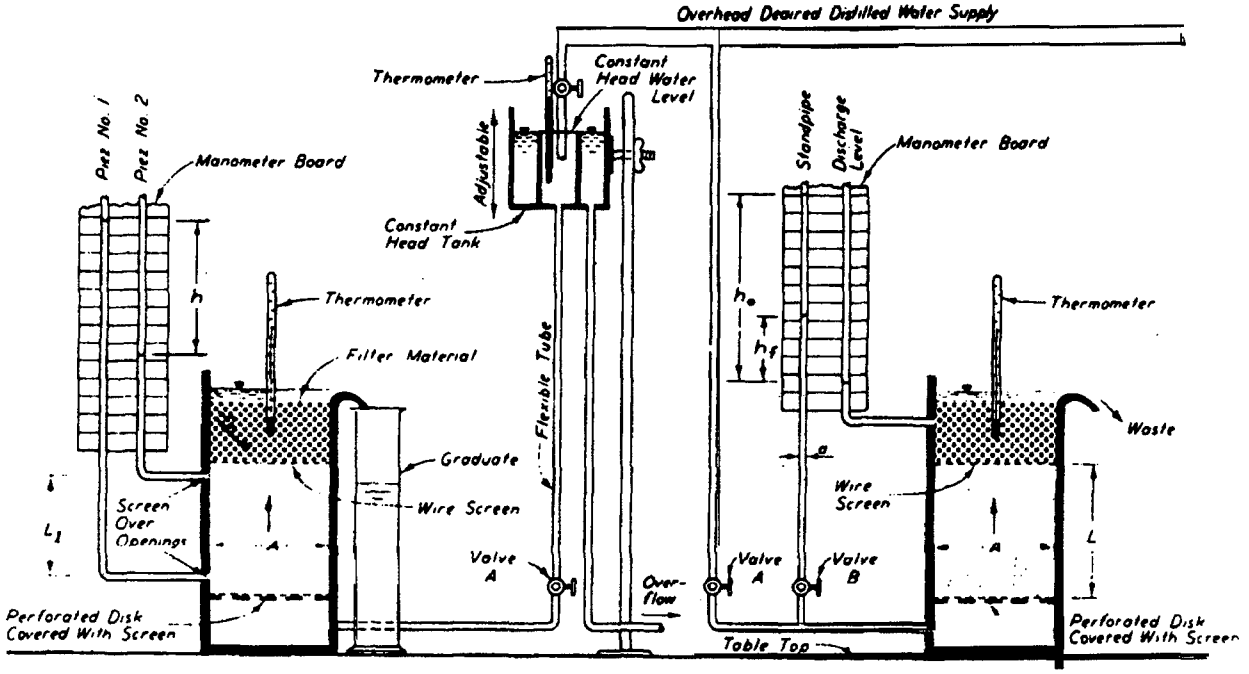
(1) Permeameter cylinders
(2) Sampling tubes
(3) Pressure cylinders
(4) Consolidometers

The permeability of remolded cohesionless soils is determined in permeameter cylinders, while the permeability of undisturbed cohesionless soils in a vertical direction can be determined using the sampling tube as a permeameter. The permeability of remolded cohesionless soils is generally used to approximate the permeability of undisturbed cohesionless soils in a horizontal direction. Pressure cylinders and consolidometers are used for fine-grained soils in the remolded, undisturbed, or compacted state. Fine-grained soils can be tested with the specimen oriented to obtain the permeability in either the vertical or horizontal direction. The above-listed devices are described in detail under the individual test procedures. Permeability tests utilizing the different types of apparatus, together with recommendations regarding their use, are discussed in the following paragraphs.

3. CONSTANT-HEAD PERMEABILITY TEST WITH PERMEAMETER CYLINDER. a. Use. The constant-head permeability test with the permeameter cylinder shall in general be used for determining the permeability of remolded samples of coarse-grained soils such as clean sands and gravels having a permeability greater than about 10×10^{-4} cm per sec.

b. Apparatus. The apparatus and accessory equipment should consist of the following:

(1) A permeameter cylinder similar to that shown schematically in Figure 3a. The permeameter cylinder should be constructed of a transparent plastic material, The inside diameter of the cylinder should be not less than about 10 times the diameter of the largest soil particles, except when the specimenis encased in a rubber membrane as in the permeability test with pressure chamber, in which case the diameter of the cylinder should be at least six times the diameter of the largest soil particles.



(a) CONSTANT-HEAD APPARATUS (b) FALLING-HEAD APPARATUS

Figure 3. Schematic diagram of constant-head and falling -head permeability apparatus

Piezometer taps along the side of the permeameter within limits to be occupied by the sample are advantageous in that the head loss within the sample is always measured across a fixed distance and rapid determination of hydraulic gradient can be made.

(2) Perforated metal or plastic disks and circular wire screens, 35 to 100 mesh, cut for a close fit inside the permeameter.

(3) Glass tubing, rubber or plastic tubing, stoppers, screw clamps, etc., necessary to make connections as shown in Figure 3a.

(4) Filter materials such as Ottawa sand, coarse sand, and gravel of various gradations.

(5) A device for maintaining a constant-head water supply.

(6) Deaired distilled† water. Tapwater contains dissolved air and gases which separate from solution in the initial layers of a test specimen of soil in the form of small bubbles. These bubbles reduce the permeability of the soil by decreasing the void space available for the flow of water. The most common method for removing dissolved air from water is by boiling the water and then cooling it at reduced pressures. This method is applicable only with small quantities of water. Freshly distilled water also has a very negligible amount of air. Large quantities of deaired distilled water may be prepared and retained for subsequent use by spraying distilled water in a fine stream into a container from which the air has been evacuated (see Fig. 4). Permeability tests on saturated specimens should show no significant decrease in permeability with time if properly deaired distilled water is used. However, if such a decrease in permeability occurs during a test, then a prefilter, consisting of a layer of the same material as the test specimen, should be used between the deaired distilled water reservoir and the test specimen to remove the air remaining in solution. ‡

† Demineralized water or tapwater when it is known to be relatively free of minerals may be used in place of distilled water.

‡ G. E. Bertram, An Experimental Investigation of Protective Filters, Soil Mechanics Series No. 7, Harvard University (Cambridge, Mass., January 1940, reprinted May 1959).

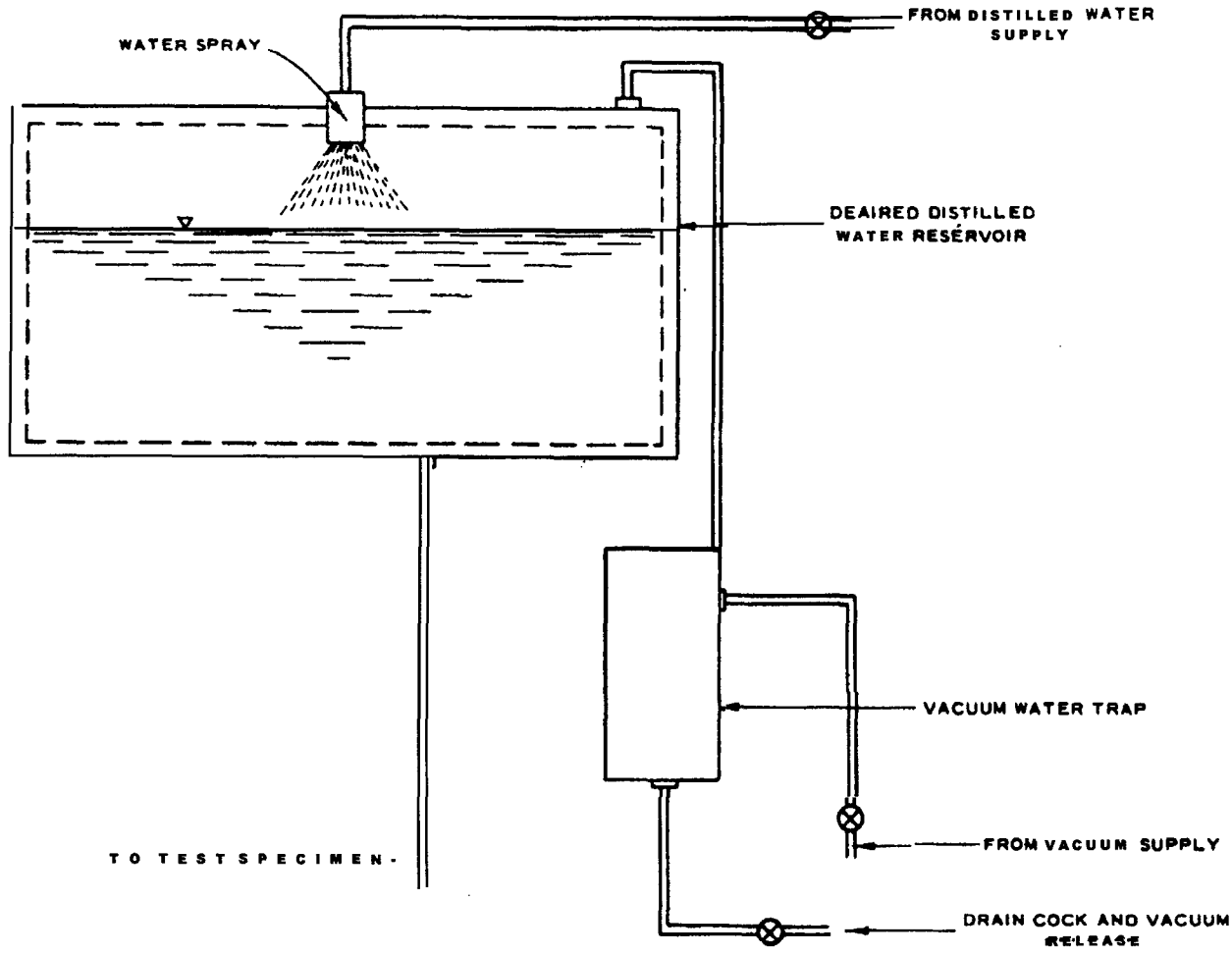


Figure 4. Schematic diagram of apparatus for preparing deaired distilled water

(7) Manometer board with tubing leading from the piezometer taps. If piezometer taps are not provided, equipment to measure the distance between the constant-head source and tailwater is required.

(8) Timing device, a watch or clock with second hand.

(9) Graduated cylinder, 100 -ml capacity.

(10) Centigrade thermometer, range 0 to 50C, accurate to 0.1 c.

(11) Balance, sensitive to 0.1 g.

(12) Oven (see Appendix I, WATER CONTENT - GENERAL).

(13) Scale, graduated in centimeters.

c. Placement and Saturation of Specimen. Placement and saturation of the specimen shall be done in the following steps:

(1) Record all identifying information for the specimen, such as project, boring number, sample number, or other pertinent data, on a data sheet (Plate VII-1 is a suggested form).

(2) Oven-dry the specimen, Allow it to cool and weigh to the nearest 0.1 g. Record the oven-dry weight of material on the data sheet opposite W_s. The amount of mate_rial should be sufficient to provide a specimen in the permeameter having a minimum length of about one to two times the diameter of the specimen.

(3) Place a wire screen, with openings small enough to retain the specimen, over a perforated disk near the bottom of the permeameter above the inlet. The screen openings should be approximately equal to the 10 percent size of the specimen.

(4) Allow deaired distilled water to enter the water inlet of the permeameter to a height of about 1/2 in. above the bottom of the screen, taking care that no air bubbles are trapped under the screen.

(5) Mix the material thoroughly and place in the permeameter to avoid segregation. The material should be dropped just at the water surface, keeping the water surface about 1/2 in. above the top of the soil during placement. A funnel or a special spoon as shown in Figure 5 is convenient for this purpose.

(6) The placement procedure outlined above will result in a saturated specimen of uniform density although in a relatively loose condition. To produce a higher density in the specimen, the sides of the permeameter containing the soil sample are tapped uniformly along its circumference and length with a rubber mallet to produce an increase in density; however, extreme caution should be exercised so that fines are not put into suspension and segregated within the sample. As an alternative to this procedure, the specimen may be placed in the in the dry using a funnel or

spoon which permits the material to fall a constant height. The desired density may be achieved by vibrating the specimen to obtain a specimen of predetermined height, Compacting the specimen in layers is not recommended as a film of dust may be formed at the surface of the compacted layer which might affect the permeability results, After placement, apply a vacuum to the top of the specimen and permit water to enter the evacuated specimen through the base of the permeameter.

(7) After the specimen has been placed, weigh the excess material, if any, and the container. The specimen weight is the difference between the original weight of sarnple and the weight of the excess material. Care must be taken so that no material is lost d· ring placement of the specimen. If there is evidence that material has been lost, oven-dry the specimen and weigh after the test as a check.

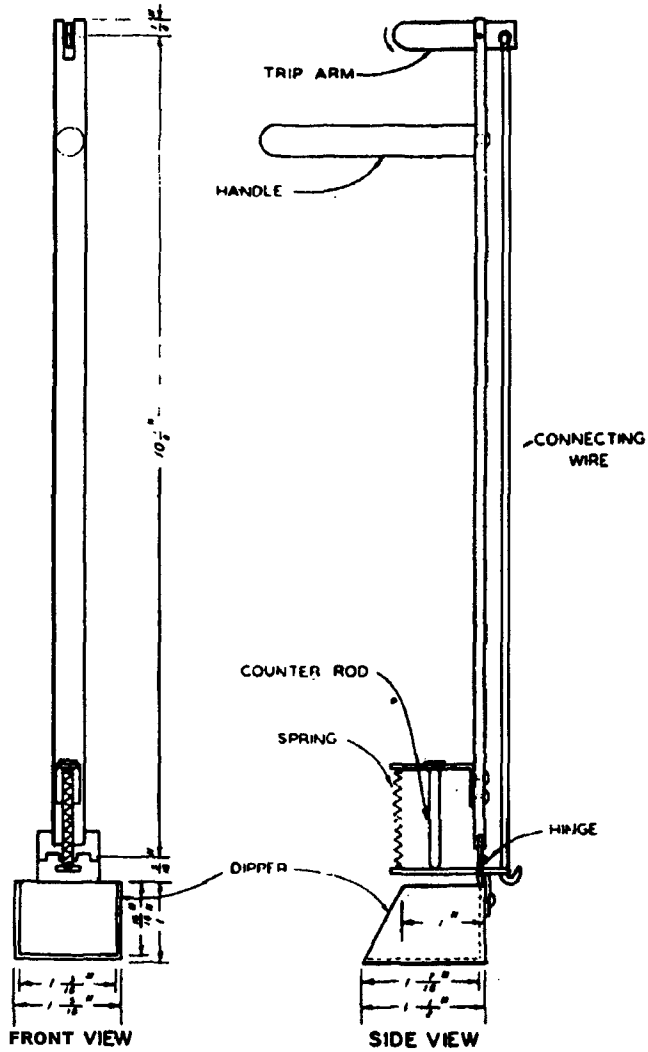


Figure 5. Spoon for placing cohesionless soils

(8) Level the top of the specimen, cover with a wire screen similar to that used at the base, and fill the remainder of the permeameter with a filter material.

(9) Measure the length of the specimen and inside diameter of the permeameter to the nearest 0.1 cm and record on the data sheet as initial height and diameter of specimen.

(10) Test the specimen at the estimated natural void ratio or

at a series of different void ratios, produced by increasing the amount of vibration after each permeability determination. Measure and record the length (height) of specimen in the permeameter prior to each determination. Permeability determinations at three different void ratios are usually sufficient to establish the relation of void ratio to permeability.

d. Procedure. The procedure shall consist of the following steps:

(1) Measure the distance, L_1, between the centers of the piezometer taps to the nearest 0.01 cm and record on the data sheet.

(2) Adjust the height of the constant-head tank to obtain the desired hydraulic gradient. The hydraulic gradient should be selected so that the flow through the specimen is laminar. The range of laminar flow conditions can be determined by plotting discharge versus hydraulic gradient. A straight-line relation indicates laminar flow, while deviations from the straight-line at high gradients indicate turbulent flow. Laminar flow for fine sands is limited to hydraulic gradients less than approximately 0.3. It is usually not practicable to achieve laminar flow for coarser soils, and the tests generally should be run at the hydraulic gradient anticipated in the field.

(3) Open valve A (see Fig. 3a) and record the initial piezometer readings after the flow has become stable. Exercise care in building up heads in the permeameter so that the specimen is not disturbed.

(4) After allowing a few minutes for equilibrium conditions to be reached, measure by means of a graduate the quantity of discharge corresponding to a given time interval. Measure the piezometric heads and the water temperature in the permeameter.

(5) Record the quantity of flow, piezometer readings, water temperature, and the time interval during which the quantity of flow was measured on the data sheet, Plate VII-1.

(6) Repeat steps (4) and (5) several times over a period of about 1 hr, and compute the coefficient of permeability corresponding to each set of measured data. If there is no substantial change in the

permeability, then the computed permeability is probably reliable. If there is a slight decrease in the permeability, then the permeability computed from the initial measurements, rather than the average, should be reported, so long as a plot of permeability versus time shows that the initial measurements are consistent with the subsequent measurements; a difference in permeability may result from a change in density caused by inadvertent jarring of the specimen in the permeameter. If there is any substantial decrease of the permeability with time, a prefilter should be used between the water reservoir and the permeameter (see paragraph 3b(6)). The criterion for judging whether a change in the computed permeability is "substantial" depends on the desired accuracy of the coefficient of permeability.

(7) If desired, reduce the void ratio as previously described and repeat the constant-head test.

e. Computations. The computations consist of the following steps:

(1) Compute the test void ratios in accordance with Appendix II, UNIT WEIGHTS, VOID RATIO, POROSITY, AND DEGREE OF SATURATION, The specific gravity shall be estimated or determined in accordance with Appendix IV, SPECIFIC GRAVITY.

(2) Compute the coefficient of permeability, k, by 'means of the following equation:

$$k_{20} = \frac{Q \times L \times R_T}{h \times A \times t}$$

where k_{20} = coefficient of permeability, cm per sec at 20 C

Q = quantity of flow, cc

L = length of specimen over which head loss is measured, cm. If piezometer taps are used, L = L_1 = distance between piezometer taps, cm

R_T = temperature correction factor for viscosity of water obtained from Table VII-1

h = loss of head in length, L, or difference in piezometer readings = $h_1 - h_2$, cm

A = cross-sectional area of specimen, sq cm

t = elapsed time, sec

Table VII-1

Correction Factor, R_T, for Viscosity of Water at Various Temperatures

Temperature Degrees C	Tenths of Degrees									
	0	1	2	3	4	5	6	7	8	9
0.0	1.783	1.777	1.771	1.765	1.759	1.753	1.747	1.741	1.735	1.729
1.0	1.723	1.717	1.711	1.705	1.699	1.694	1.688	1.682	1.676	1.670
2.0	1.664	1.659	1.654	1.648	1.643	1.638	1.632	1.627	1.622	1.616
3.0	1.611	1.606	1.601	1.596	1.590	1.585	1.580	1.575	1.570	1.565
4.0	1.560	1.555	1.550	1.545	1.540	1.535	1.531	1.526	1.521	1.516
5.0	1.511	1.507	1.502	1.498	1.493	1.488	1.484	1.479	1.475	1.470
6.0	1.465	1.461	1.457	1.452	1.448	1.443	1.439	1.435	1.430	1.426
7.0	1.421	1.417	1.413	1.409	1.404	1.400	1.396	1.392	1.388	1.383
8.0	1.379	1.375	1.371	1.367	1.363	1.359	1.355	1.351	1.347	1.343
9.0	1.339	1.336	1.332	1.328	1.324	1.320	1.317	1.313	1.309	1.305
10.0	1.301	1.298	1.294	1.290	1.287	1.283	1.279	1.276	1.272	1.269
11.0	1.265	1.262	1.258	1.255	1.251	1.248	1.244	1.241	1.237	1.234
12.0	1.230	1.227	1.223	1.220	1.217	1.213	1.210	1.207	1.203	1.200
13.0	1.197	1.194	1.190	1.187	1.184	1.181	1.178	1.175	1.171	1.168
14.0	1.165	1.162	1.159	1.156	1.153	1.150	1.147	1.144	1.141	1.138
15.0	1.135	1.132	1.129	1.126	1.123	1.120	1.117	1.114	1.111	1.108
16.0	1.106	1.103	1.100	1.097	1.094	1.091	1.089	1.086	1.083	1.080
17.0	1.077	1.075	1.072	1.069	1.067	1.064	1.061	1.059	1.056	1.053
18.0	1.051	1.048	1.045	1.043	1.040	1.038	1.035	1.033	1.030	1.027
19.0	1.025	1.022	1.020	1.017	1.015	1.012	1.010	1.007	1.005	1.002
20.0	1.000	0.998	0.995	0.993	0.990	0.988	0.986	0.983	0.981	0.979
21.0	0.976	0.974	0.972	0.969	0.967	0.965	0.962	0.960	0.958	0.955
22.0	0.953	0.951	0.949	0.947	0.944	0.942	0.940	0.938	0.936	0.933
23.0	0.931	0.929	0.927	0.925	0.923	0.920	0.918	0.916	0.914	0.912
24.0	0.910	0.908	0.906	0.904	0.901	0.899	0.897	0.895	0.893	0.891
25.0	0.889	0.887	0.885	0.883	0.881	0.879	0.877	0.875	0.873	0.871
26.0	0.869	0.867	0.866	0.864	0.862	0.860	0.858	0.856	0.854	0.852
27.0	0.850	0.848	0.847	0.845	0.843	0.841	0.839	0.837	0.836	0.834
28.0	0.832	0.830	0.828	0.826	0.825	0.823	0.821	0.819	0.818	0.816
29.0	0.814	0.812	0.810	0.809	0.807	0.805	0.804	0.802	0.800	0.798
30.0	0.797	0.795	0.793	0.792	0.790	0.788	0.787	0.785	0.783	0.782
31.0	0.780	0.778	0.777	0.775	0.774	0.772	0.770	0.769	0.767	0.766
32.0	0.764	0.763	0.761	0.759	0.758	0.756	0.755	0.753	0.752	0.750
33.0	0.749	0.747	0.746	0.744	0.743	0.741	0.739	0.738	0.736	0.735
34.0	0.733	0.732	0.731	0.729	0.728	0.726	0.725	0.723	0.722	0.720
35.0	0.719	0.718	0.716	0.715	0.713	0.712	0.711	0.709	0.708	0.706
36.0	0.705	0.704	0.702	0.701	0.699	0.698	0.697	0.695	0.694	0.693
37.0	0.691	0.690	0.689	0.687	0.686	0.685	0.683	0.682	0.681	0.679
38.0	0.678	0.677	0.675	0.674	0.673	0.672	0.670	0.669	0.668	0.666
39.0	0.665	0.664	0.663	0.661	0.660	0.659	0.658	0.656	0.655	0.654
40.0	0.653	0.652	0.650	0.649	0.648	0.647	0.646	0.644	0.643	0.642
41.0	0.641	0.639	0.638	0.637	0.636	0.635	0.634	0.632	0.631	0.630
42.0	0.629	0.628	0.627	0.626	0.624	0.623	0.622	0.621	0.620	0.619
43.0	0.618	0.616	0.615	0.614	0.613	0.612	0.611	0.610	0.609	0.608
44.0	0.607	0.606	0.604	0.603	0.602	0.601	0.600	0.599	0.598	0.597
45.0	0.596	0.595	0.594	0.593	0.592	0.591	0.590	0.588	0.587	0.586
46.0	0.585	0.584	0.583	0.582	0.581	0.580	0.579	0.578	0.577	0.576
47.0	0.575	0.574	0.573	0.572	0.571	0.570	0.569	0.568	0.567	0.566
48.0	0.565	0.564	0.564	0.563	0.562	0.561	0.560	0.559	0.558	0.557
49.0	0.556	0.555	0.554	0.553	0.552	0.551	0.550	0.549	0.548	0.548

Computed from Table 170 - Smithsonian Physical Tables - 8th Edition
Correction factor, R_T, is found by dividing the viscosity of water at the test temperature by the viscosity of water at 20 C.

f. Presentation of Results. The coefficient of permeability shall be reported in units with coefficients of $1.0, 1 \times 10^{-4}$, and 1×10^{-9} cm per sec. The void ratio of the specimen shall be reported with all values of k. The coefficient of permeability, k, is logarithmically dependent upon the void ratio of the soil. Where k is determined at several void ratios, the test results shall be presented on a semilogarithmic chart as shown in Figure 6 in which k is plotted on the abscissa (logarithmic scale) and the void ratio is plotted on the ordinate (arithmetic scale).

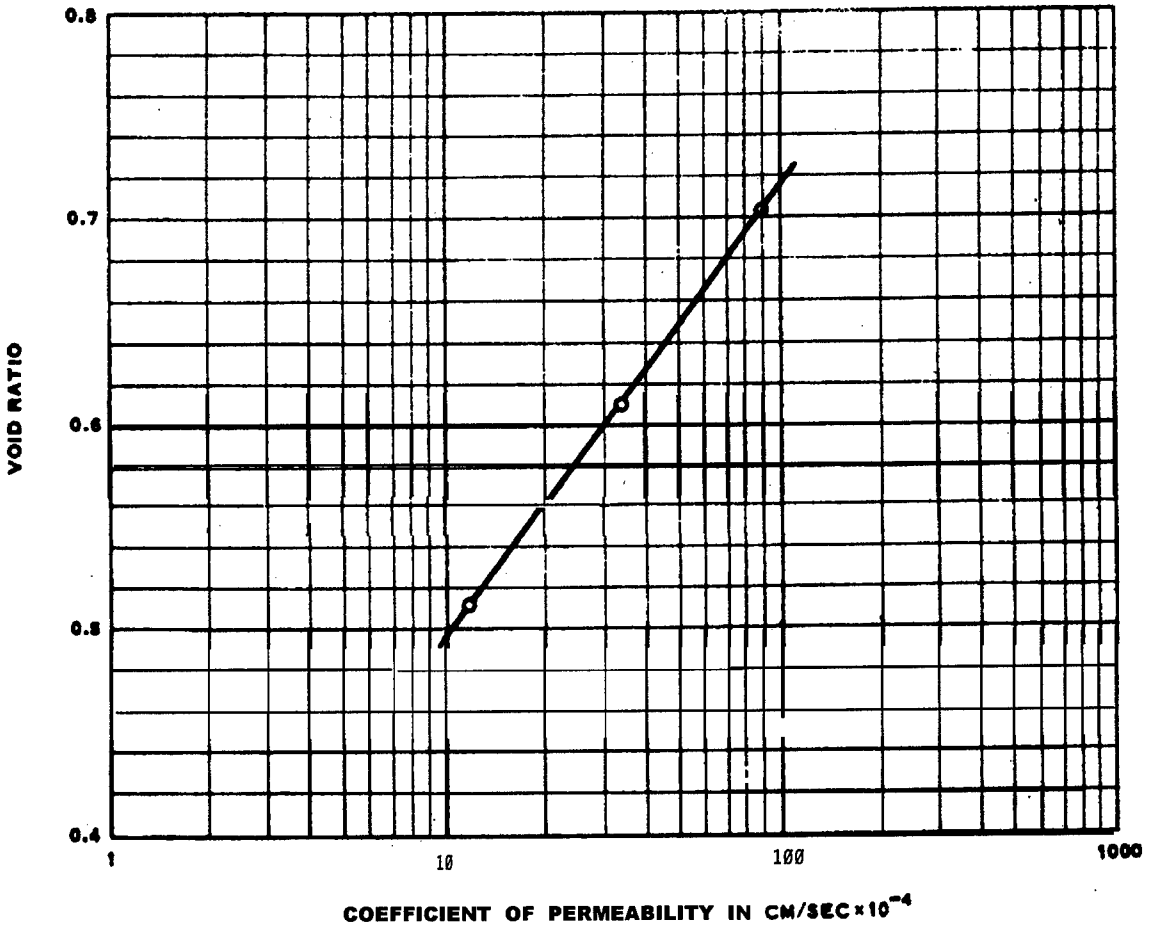


Figure 6. Relation between permeability and void ratio for cohesionless soils

4. FALLING-HEAD PERMEABILITY TEST WITH PERMEAMETER CYLINDER, a. Use. The falling-head test with the permeameter

cylinder should in general be used for determining the permeability of remolded samples of cohesionless soils having a permeability less than about 10×10^{-4} *cm* per sec.

b. Apparatus. The apparatus and accessory equipment should consist of the following:

(1) A permeameter cylinder similar to that shown schematically in Figure 3b, or modified versions thereof. The permeameter cylinder should be constructed of a transparent plastic material. The inside diameter of the cylinder should be not less than about 10 times the diameter of the largest soil particles. The use of two piezometer taps, as shown by Figure 3b, connected to a standpipe and discharge level tube eliminates the necessity for taking into account the height of capillary rise which would be necessary in the case of a single standpipe of small size. The height of capillary rise for a given tube and condition can be measured simply by standing the tube upright in a beaker full of water. The size of standpipe to be used is generally based on experience with the equipment used and soils tested. In order to accelerate testing, air pressure may be applied to the standpipe to increase the hydraulic gradient.

(2) Perforated metal or plastic disks and circular wire screens, 35 to 100 mesh, cut for a close fit inside the permeameter.

(3) Glass tubing, rubber or plastic tubing, stoppers, screw clamps, etc., necessary to make connections as shown in Figure 3b.

(4) Filter materials such as Ottawa sand, coarse sand, and gravel of various gradations.

(5) Deaired distilled water, prepared according to paragraph 3b(6).

(6) Manometer board or suitable scales for measuring levels in piezometers or standpipe.

(7) Timing device, a watch or clock with second hand.

(8) Centigrade thermometer, range 0 to 50 C, accurate to 0.1 C.

(9) Balance, sensitive to 0.1 g.

(10) Oven (see Appendix I, WATER CONTENT - GENERAL).

(if) Scale, graduated in centimeters.

c. Placement and Saturation of Specimen. Placement and saturation of the specimen shall be done as described in paragraph 3c. Identifying information for the sample and test data shall be entered *on* a data sheet similar to Plate VII-2.

d. Procedure. The procedure shall consist of the following steps:

(1) Measure and record the height of the specimen, L, and the cross-sectional area of the specimen, A.

(2) With valve B open (see Fig. 3b), crack valve A and slowly bring the water level up to the discharge level of the permeameter.

(3) Raise the head of water in the standpipe above the discharge level of the permeameter. The difference in head should not result in an excessively high hydraulic gradient during the test. Close valves A and B.

(4) Begin the test by opening valve B. Start the timer. As the water flows through the specimen, measure and record the height of water in the standpipe above the discharge level, h_o, in centimeters, at time t_o, and the height of water above the discharge level, h_f, in centimeters, at time t_f.

(5) Observe and record the temperature of the water in the permeameter.

(6) Repeat the determination of permeability, and if the computed values differ by an appreciable amount, repeat the test until consistent values of permeability are obtained.

e. Computations. The computations consist of the following steps:

(1) Compute the test void ratios as outlined in paragraph 3e(1).

(2) Compute the coefficient of permeability, k, by means of the following equation:

$$k = 2.303 \frac{a}{A} \frac{L}{t} \left(\log \frac{h_o}{h_f} \right) R_T$$

where a = inside area of standpipe, sq cm

A = cross-sectional area of specimen, sq cm

L = length of specimen, cm

t = elapsed time ($t_f - t_o$), sec

h_o = height of water in standpipe above discharge level at time t_o, cm

h_f = height of water in standpipe above discharge level at time t_f, cm

R_T = temperature correction factor for viscosity of water obtained from Table VII-1, degrees C

If a single standpipe of small diameter is used as shown in Figure 2, the height of capillary rise, h_c, should be subtracted from the standpipe readings to obtain h_o and h_f.

f. Presentation of Results. The results of the falling-head permeability test shall be reported as described in paragraph 3 f.

5. PERMEABILITY TESTS WITH SAMPLING TUBES. Permeability tests may be performed directly on undisturbed samples without removing them from the sampling tubes. The sampling tube serves as the permeameter cylinder. The method is applicable primarily to cohesionless soils which cannot be removed from the sampling tube without excessive disturbance. The permeability obtained is in the direction in which the sample was taken, i.e. generally vertical. The permeability obtained in a vertical direction may be substantially less than that obtained in a horizontal direction.

Permeability tests with sampling tubes may be performed under constant-head or falling-head conditions of flow, depending on the estimated permeability of the sample (see paragraph 2a). The equipment should be capable of reproducing the conditions of flow in the constant-head or falling-head tests. It is important that all disturbed material or material containing drilling mud be removed from the top and bottom of the sample. The ends of the sample should be protected by screens held in place by perforated packers. The test procedure and computations are

the same as those described previously for each test.

6. PERMEABILITY TEST WITH PRESSURE CHAMBER. In the permeability test with a pressure chamber, see Figure 7, a cylindrical specimen is confined in a rubber membrane and subjected to an external hydrostatic pressure during the permeability test. The advantages of this type of test are: (a) leakage along the sides of the specimen, which would occur if the specimen were tested in a permeameter, is prevented, and (b) the specimen can be tested under conditions of loading expected in the field. The test is applicable primarily to cohesive soils in the undisturbed, remolded or compacted state. Complete saturation of the specimen, if it is not fully saturated initially, is practically impossible. Consequently, this test should be used only for soils that are fully saturated, unless values of permeability are purposely desired for soils in an unsaturated condition. The permeability test with the pressure chamber is usually performed as a falling-head test.

The permeability specimens for use in the pressure chamber generally should be 2.8 in. in diameter, as rubber membranes and equipment for cutting and trimming specimens of this size are available for triaxial testing apparatus (see Appendix X, TRIAXIAL COMPRESSION TESTS). A specimen length of about 4 in. is adequate. (The dimensions of a test specimen may be varied if equipment and supplies are available to make a suitable test setup.) The pressure in the chamber should not be less than the maximum head on the specimen during the test. The other test procedure and computations are the same as those described for the falling-head test. The linear relation between permeability and void ratio on a semilogarithmic plot as shown in Figure 6 is usually not applicable to fine-grained soils, particularly when compacted. Other methods of presenting permeability-void ratio data may be desirable.

7. PERMEABILITY TESTS WITH BACK PRESSURE.

a. Description. Gas bubbles in the pores of a compacted or undisturbed specimen of fine-grained soil will invalidate the results of the

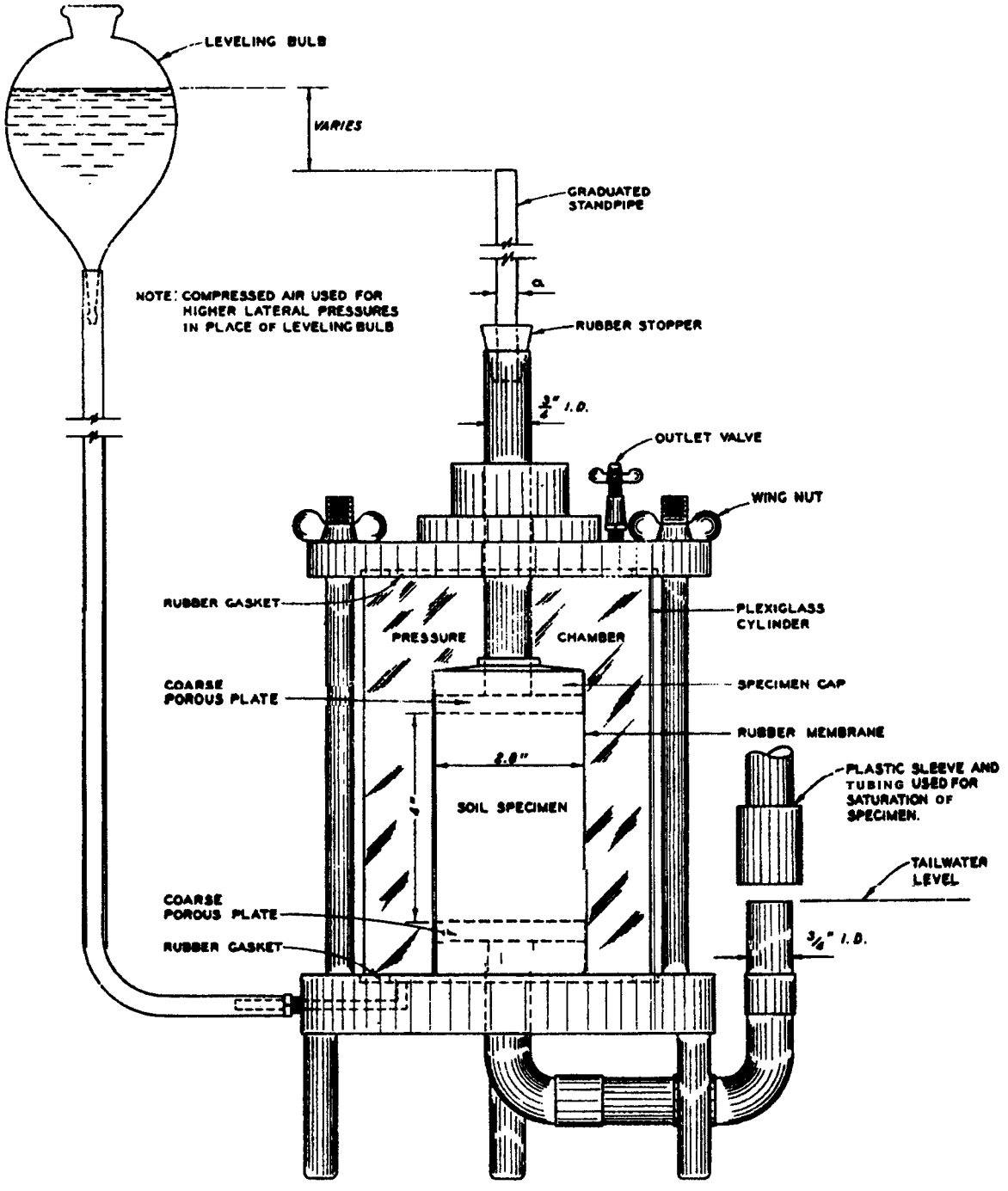


Figure 7. Pressure chamber for permeability test

permeability tests described in the preceding paragraphs. It is known that an increase in pressure will cause a reduction in volume of gas bubbles and also an increased weight of gas dissolved in water. To each degree of saturation there corresponds a certain additional pressure (back pressure) which, if applied to the pore fluid of the specimen, will cause complete saturation. The permeability test with back pressure is performed in a pressure chamber such as that shown in Figure 8, utilizing equipment that permits increasing the chamber pressure and pore pressure simultaneously, maintaining their difference constant The method is generally applicable to fine-grained soils that are not fully saturated. Apparatus and procedures have been described by A. Casagrande† and L. Bjerrum and J. Huder.‡

b. Procedure (see Fig. 8). The procedure shall, consist of the following steps :

(1) After having determined the dimensions and wet weight of the test specimen, place it in the triaxial apparatus, using the same procedure as for setting up a specimen for an R triaxial test with pore pressure measurements except that filter strips should not be used (see para 7, APPENDIX X, TRIAXIAL COMPRESSION TESTS).

(2) Saturate the specimen and verify 100 percent saturation using the procedure described in paragraph 7b, APPENDIX X, TRIAXIAL COMPRESSION TESTS. Burette "A" is utilized during this operation.

(3) With the drainage valves closed, increase the chamber

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† Casagrande, A., "Third Progress Report on Investigation of Stress Deformation and Strength Characteristics of Compacted Clays," Soil Mechanics Series No. 70, Nov 1963, Harvard University, Cambridge, Mass., pp 30 and 31.

‡ Bjerrum, L. and Huder, J., "Measurement of the Permeability of Compacted Clays ," Proceedings, Fourth International Conference on Soil Mechanics and Foundation Engineering, London, Vol 1, Aug 1957, pp 6-8.

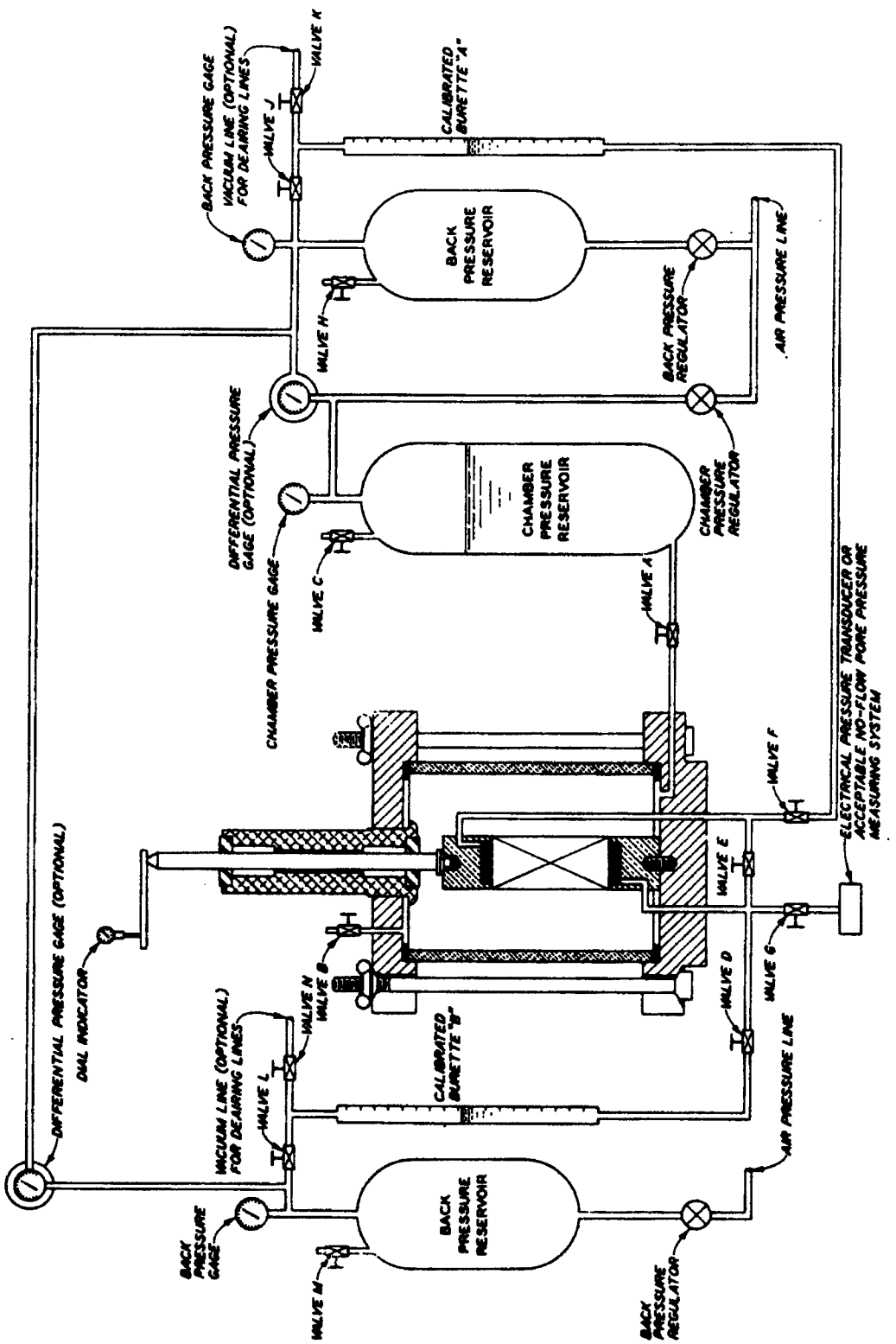


Figure 8. Schematic diagram of typical triaxial compression apparatus for permeability tests with back pressure

pressure to attain the desired effective consolidation pressure (chamber pressure minus back pressure). At zero elapsed time, open valves E and F.

(4) Record time, dial indicator reading, and burette reading at elapsed times of 0, 15, and 30 sec, 1, 2, 4, 8, and 15 min, and 1, 2, 4, and 8 hr, etc. Plot the dial indicator readings and burette readings on an arithmetic scale versus elapsed time on a log scale. When the consolidation curves indicate that primary consolidation is complete close valves E and F.

(5) Apply a pressure to burette B greater than that in burette A. The difference between the pressures in burettes B and A is equal to the head loss h; h divided by the height of the specimen after consolidation, L, is the hydraulic gradient. The difference between the two pressures should be kept as small as practicable, consistent with the requirement that the rate of flow be large enough to make accurate measurements of the quantity of flow within a reasonable period of time. Because the difference in the two pressures may be very small in comparison to the pressures at the ends of the specimen, and because the head loss must be maintained constant throughout the test, the difference between the pressures within the burettes must be measured accurately; a differential pressure gage is very useful for this purpose. The difference between the elevations of the water within the burettes should also be considered (1 in. of water = 0.036 psi of pressure).

(6) Open valves D and F. Record the burette readings at any zero elapsed time. Make readings of burettes A and B and of temperature at various elapsed times (the interval between successive readings depends upon the permeability of the soil and the dimensions of the specimen). Plot arithmetically the change in readings of both burettes versus time. Continue making readings until the two curves become parallel and straight over a sufficient length of time to accurately determine the rate of flow (slope of the curves).

(7) If it is desired to determine the permeability at several void ratios, steps 3 through 6 can be repeated, using different consolidation pressures in step 3.

(8) At the end of the permeability determinations, close all drainage valves and reduce the chamber pressure to zero; disassemble the apparatus.

(9) Determine the wet and dry weights of the specimen.

c. Computations. The computations consist of the following steps.

(1) Compute the test void ratios as outlined in paragraph 3e(1).

(2) Computations of coefficients of permeability are the same as those described for the constant-head permeability test.

8. PERMEABILITY TESTS WITH CONSOLIDOMETER. A permeability test in a consolidometer (see Appendix VIII, CONSOLIDATION TEST) is essentially similar to that conducted in a pressure chamber, except that the specimen is placed within a relatively rigid ring and is loaded vertically. The test can be used as an alternate to the permeability test in the pressure chamber. The test is applicable primarily to cohesive soils in a fully saturated condition. Testing is usually performed under falling-head conditions.

A schematic diagram of the consolidation apparatus set up for a falling-head permeability test is shown in Figure 9. Identifying information for the specimen and subsequent test data are entered on a data sheet (Plate VII-3 is a suggested form). The specimen should be placed in the specimen ring and the apparatus assembled as outlined under Appendix VIII, CONSOLIDATION TEST. The specimen is consolidated under the desired load and the falling-head test is performed as previously described. The

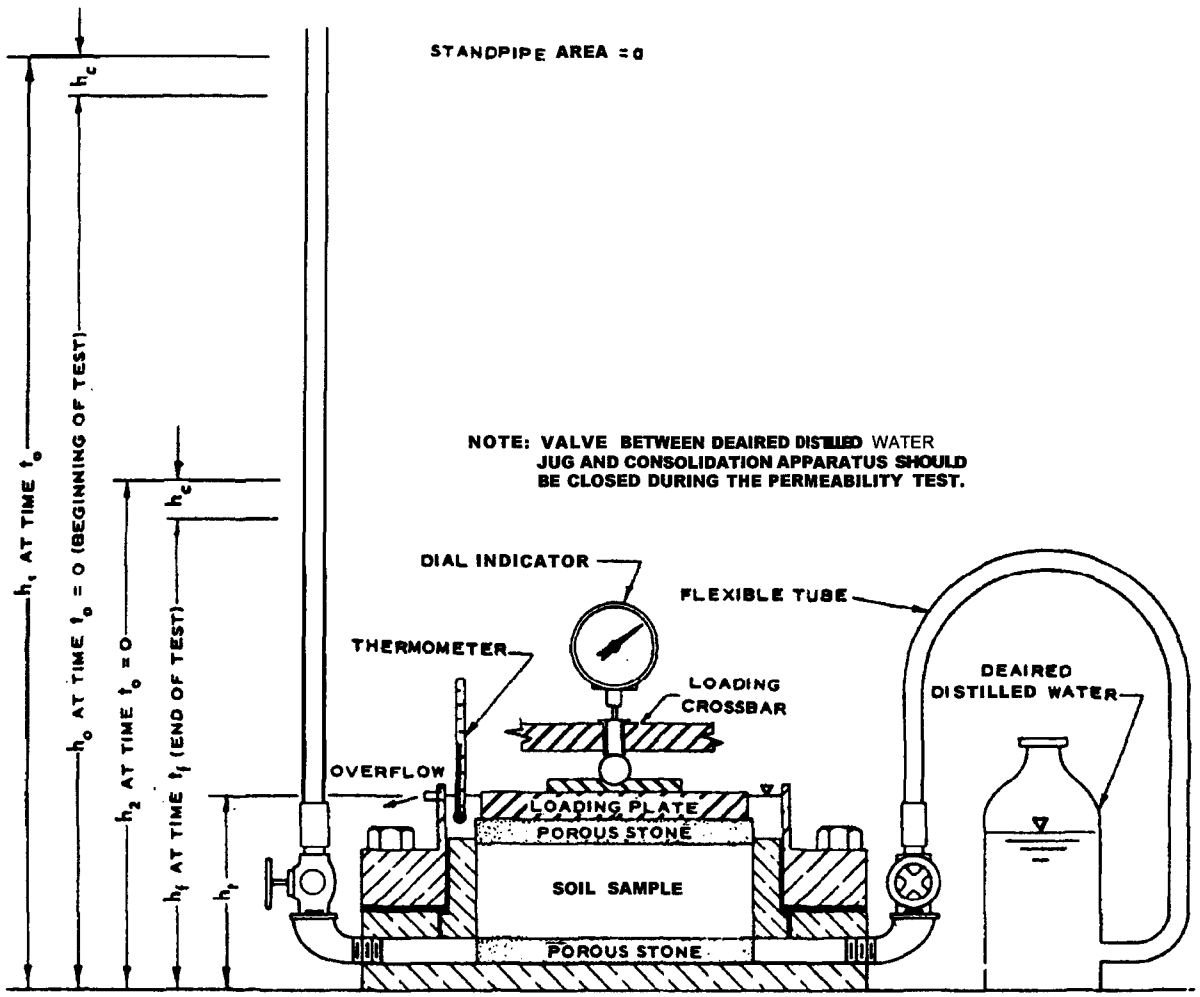


Figure 9. Schematic diagram of falling-head device for permeability test in consolidometer

net head on the specimen may be increased by use of air pressure; however, the pressure on the pore water should not exceed 25 to 30 percent of the vertical pressure under which the specimen has consolidated. Dial indicator readings are observed before and after consolidation to permit computation of void ratios. The determination of the coefficient of permeability may be made in conjunction with the consolidation test, in which case the test is performed at the end of the consolidation phase under each load increment. Computations are similar to those described for the

falling-head test with the permeameter cylinder.

The permeability may also be determined indirectly from computations using data obtained during the consolidation test; however the assumptions on which the method is based are seldom satisfied, and consequently, the direct determination of permeability should be employed where reliable values of permeability are required.

9. POSSIBLE ERRORS. Following are possible errors that would cause inaccurate determinations of the coefficient of permeability:

a. Stratification or nonuniform compaction of cohesionless soils. If the specimen is compacted in layers, any accumulation of fines at the surface of the layers will reduce the measured coefficient of permeability.

b. Incomplete initial saturation of specimen.

c. Excessive hydraulic gradient. Darcy's law is applicable only to conditions of laminar flow.

d. Air dissolved in water. No other source of error is as troublesome as the accumulation of air in the specimen from the flowing water. As water enters the specimen, small quantities of air dissolved in the water will tend to collect as fine bubbles at the soil-water interface and reduce the permeability at this interface with increasing time. The method for detecting and avoiding this problem is described in paragraph 3d(6). (It should be noted that air accumulation will not affect the coefficient of permeability determined by the constant-head test if piezometer taps along the side of the specimen are used to measure the head loss.)

e. Leakage along side of specimen in permeameter. One major advantage to the use of the triaxial compression chamber for permeability tests (see paragraphs 6 and 7) is that the specimen is confined by a flexible membrane which is pressed tightly against the specimen by the chamber pressure.

CONSTANT-HEAD PERMEABILITY TEST

DATE _____

PROJECT _____

BORING NO. _____

Sample or Specimen No.

Wt in grams						
	Tare plus dry soil			Diameter of specimen, cm	D	
	Tare			Area of specimen, sq cm	A	
	Dry soil	W_s		Initial height of specimen, cm	L	
	Specific gravity	G		Initial vol of spec, cc = AL	V	
	Vol of solids, cc = $W_s \div G$	V_s		Initial void ratio = $(V - V_s) \div V_s$	e	

Distance between piezometer taps, cm L_1 _____

Test No.		1		2		3	
Height of specimen, cm	L						
Void ratio = $(AL - V_s) \div V_s$	e						
		1a	1b	2a	2b	3a	3b
Reading of piez 1, cm	h_1						
Reading of piez 2, cm	h_2						
Head loss, cm = $h_1 - h_2$	h						
Quantity of flow, cc	Q						
Elapsed time, sec	t						
Water temperature, °C	T						
Viscosity correction factor(1)	R_T						
Coefficient of permeability,(2) cm/sec	k_{20}						
	Avg						

(1) Correction factor for viscosity of water at 20 C obtained from table VII-1.

(2) $k_{20} = \dfrac{Q \times L \times R_T}{h \times A \times t}$.

where L = height of specimen or distance between piezometer taps if used.

Remarks _____

Technician _____ Computed by _____ Checked by _____

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PLATE VII-1

FALLING-HEAD PERMEABILITY TEST

DATE _____

PROJECT _____

BORING NO. _____

SAMPLE OR SPECIMEN NO.

WT IN GRAMS	TARE PLUS DRY SOIL			DIAMETER OF SPECIMEN, CM	D
	TARE			AREA OF SPECIMEN, SQ CM	A
	DRY SOIL	W_s		INITIAL HEIGHT OF SPECIMEN, CM	L
SPECIFIC GRAVITY		G		INITIAL VOL OF SPEC, CC = AL	V
VOL OF SOLIDS, CC = $W_s \div G$		V_s		INITIAL VOID RATIO = $(V - V_s) \div V_s$	e
AREA OF STANDPIPE, SQ CM		a		CONSTANT = (2.303 × a) ÷ A	C

TEST NO.		1		2		3	
HEIGHT OF SPECIMEN, CM	L						
VOID RATIO = $(AL - V_s) \div V_s$	e						
		1a	1b	2a	2b	3a	3b
INITIAL TIME	t_o						
FINAL TIME	t_f						
ELAPSED TIME, SEC = $t_f - t_o$	t						
INITIAL HEAD, CM	h_o						
FINAL HEAD, CM	h_f						
LOG $(h_o \div h_f)$							
WATER TEMPERATURE, °C	T						
VISCOSITY CORRECTION FACTOR[1]	R_T						
COEFFICIENT OF PERMEABILITY,[2] CM/SEC	k_{20}						
	AVG						

(1) CORRECTION FACTOR FOR VISCOSITY OF WATER AT 20 C OBTAINED FROM TABLE VII-1.

(2) $k_{20} = 2.303 \frac{a}{A} \frac{L}{t} \text{LOG} \frac{h_o}{h_f} \times R_T = \frac{CL}{t} \left(\text{LOG} \frac{h_o}{h_f} \right) R_T$

REMARKS _____

TECHNICIAN _____ COMPUTED BY _____ CHECKED BY _____

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PLATE VII-2

FALLING-HEAD PERMEABILITY TEST WITH CONSOLIDOMETER

DATE _____

PROJECT _____

BORING NO. _____

SAMPLE OR SPECIMEN NO.

WT IN GRAMS	TARE PLUS DRY SOIL			DIAMETER OF SPECIMEN, CM	D	
	TARE			AREA OF SPECIMEN, SQ CM	A	
	DRY SOIL	W_s		INITIAL HEIGHT OF SPECIMEN, CM	L	
SPECIFIC GRAVITY		G_s		INITIAL VOL OF SPEC, CC = AL	V	
VOL OF SOLIDS, CC = $W_s \div G_s$		V_s		INITIAL VOID RATIO = $(V - V_s) \div V_s$	e	
AREA OF STANDPIPE, SQ CM		a		CONSTANT = (2.303 × a) ÷ A	C	
CAPILLARY RISE, CM		h_c		INITIAL DIAL READING, IN.	D_o	
HEIGHT OF TAILWATER, CM		h_t		CORRECTED TAILWATER, CM, $h_t + h_c$	Δh	

TEST NO.		1		2		3	
LOAD INCREMENT, T/SQ FT	P						
DIAL READING AT START, IN.	D_1						
CHANGE IN HT OF SPEC, IN. = $D_o - D_1$	ΔD						
HT OF SPEC, CM = L − 2.54 ΔD	L						
VOID RATIO = $(AL - V_s) \div V_s$	e						
		1a	1b	2a	2b	3a	3b
INITIAL TIME	t_o						
FINAL TIME	t_f						
ELAPSED TIME, SEC = $t_f - t_o$	t						
INITIAL HEIGHT, CM	h_1						
FINAL HEIGHT, CM	h_2						
WATER TEMPERATURE, °C	T						
VISCOSITY CORRECTION FACTOR[1]	R_T						
COEFFICIENT OF PERMEABILITY,[2] CM/SEC	k_{20}						
	AVG						

(1) CORRECTION FACTOR FOR VISCOSITY OF WATER AT 20 C OBTAINED FROM TABLE VII-1.

(2) $k_{20} = 2.303 \frac{a}{A} \frac{L}{t} \left(\text{LOG} \frac{h_1 - \Delta h}{h_2 - \Delta h} \right) R_T = \frac{CL}{t} \left(\text{LOG} \frac{h_1 - \Delta h}{h_2 - \Delta h} \right) R_T$

REMARKS _____

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PLATE VII-3

APPENDIX VIII:

CONSOLIDATION TEST

1. INTRODUCTION. Consolidation is the process of gradual transfer of an applied load from the pore water to the soil structure as pore water is squeezed out of the voids. The amount of water that escapes depends on the size of the load and compressibility of the soil. The rate at which it escapes depends on the coefficient of permeability, thickness, and compressibility of the soil. The rate and amount of consolidation with load are usually determined in the laboratory by the one-dimensional consolidation test. In this test, a laterally confined soil is subjected to successively increased vertical pressure, allowing free drainage from the top and bottom surfaces.

2. APPARATUS. The apparatus should consist of the following:

a. Consolidometer shall consist of a rigid base, a consolidation ring, porous stones, a rigid loading plate, and a support for a dial indicator (Fig. 1). It may be either the fixed-ring or the floating -ring type both of which are shown in Figure 2.

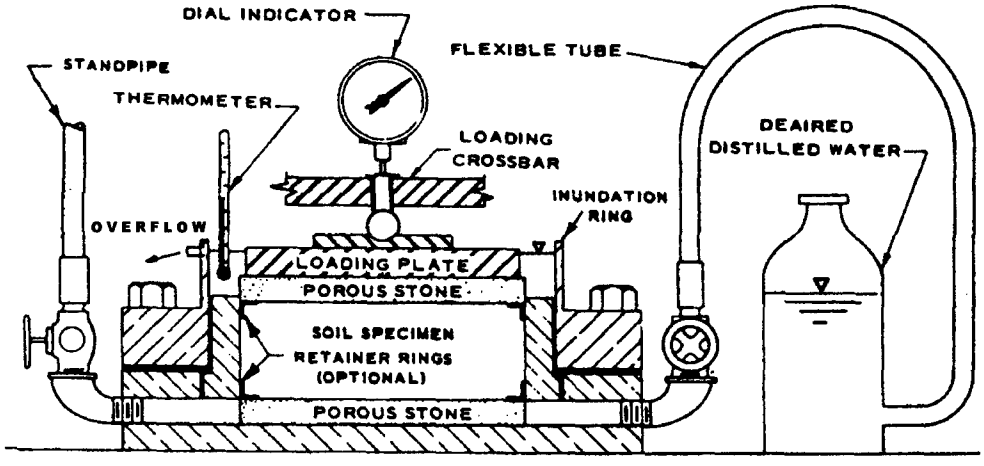
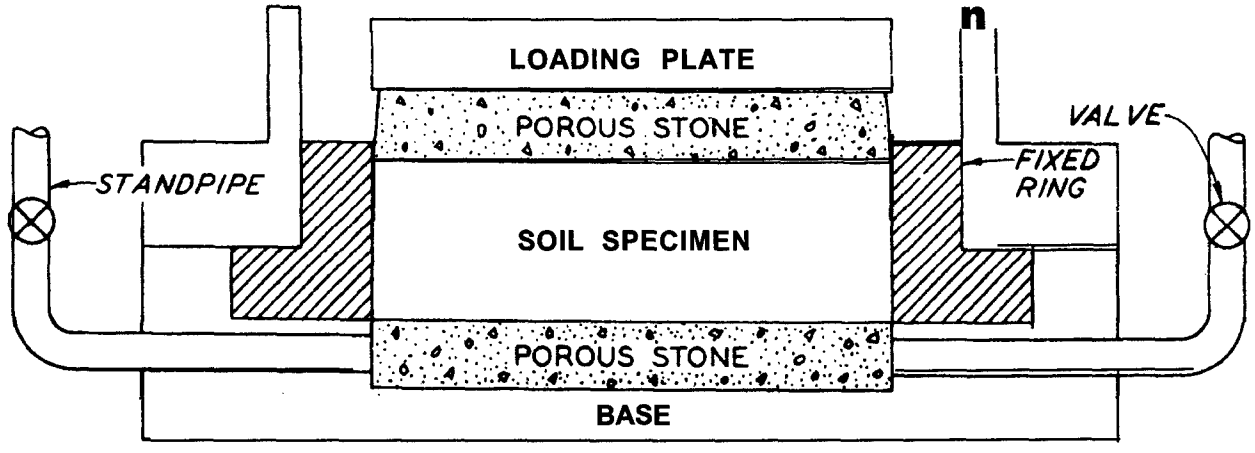
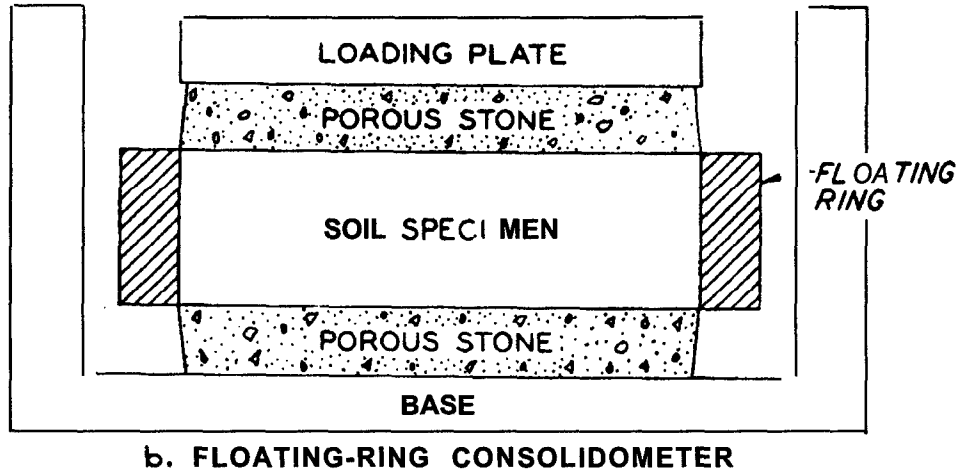


Figure 1. Typical consolidometer



a. FIXED-RING CONSOLIDOMETER



b. FLOATING-RING CONSOLIDOMETER

Figure 2. Schematic diagrams of fixed-ring and floating-ring consolidometers

The various metal parts of the consolidometer shall be of the same non-corrosive material. All-plastic or combination plastic and metal consolidometers may also be used to reduce electrochemical effects. The consolidometer shall conform to the following requirements.

(1) Fixed-ring consolidometer shall have a rigid base with a recess for supporting the bottom porous stone and for seating and attaching the consolidation ring. The upper surface of the'recess shall be grooved to permit drainage. The base shall also have (a) an inundation

ring to permit submergence of the specimen in water to prevent evaporation of water from the specimen during the test, and (b) suitable connections and a standpipe for making permeability tests.

(2) Floating-ring consolidometer† shall have a rigid base for supporting the bottom porous stone. The base shall be large enough to permit free vertical movement of the consolidation ring and shall have a chamber surrounding the ring for submergence of the specimen.

b. Consolidation ring shall completely and rigidly confine and support the specimen laterally. The inside diameter of the ring should not be less than 2-3/4 in. and preferably not less than 4 in.; use of larger rings for specimens of larger diameter, particularly with the fixed-ring consolidometer, will reduce the percentage of applied load carried by side friction and consequently will provide more accurate results. Normally, the ratio of the height of ring to inside diameter of ring should be between 1/4 and 1/6. The consolidation ring may be lined with a material such as Teflon to reduce the friction between the ring and a specimen of fine-grained soil. A stainless steel ring is preferable for specimens containing abrasive particles.

c. Porous stones more pervious than the specimen of soil should be used to permit effective drainage. For routine testing, stones of medium porosity are satisfactory. The diameter of the porous stones

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† In the floating-ring consolidometer the friction between the inside of the ring and the specimen is less than that in the fixed-ring type. However, when very soft soils are tested with the floating-ring consolidometer, the side friction will not support the weight of the ring, and compression occurs toward the middle of the specimen from top and bottom. The floating-ring device is suitable only for comparatively stiff soils, and has the disadvantage that it cannot be used for permeability tests.

shall be such as to prevent the squeezing out of soil through the clearance spaces between the ring and stone and to permit free compression of the specimen without binding; to minimize the possibility of binding, the sides of the upper porous stone of the fixed-ring consolidometer should be slightly tapered away from the specimen, while both porous stones of the floating-ring consolidometer should be tapered. A clearance of about 0.010 to 0.015 in. around the stone generally will be adequate; however, if very soft soils are tested, a smaller clearance may be desirable or retainer rings may be used as shown in Figure 1. Details of a typical retainer ring are shown in Figure 3. The porous stones should be cleaned after every test, preferably in an ultrasonic cleaner or by boiling and flushing.

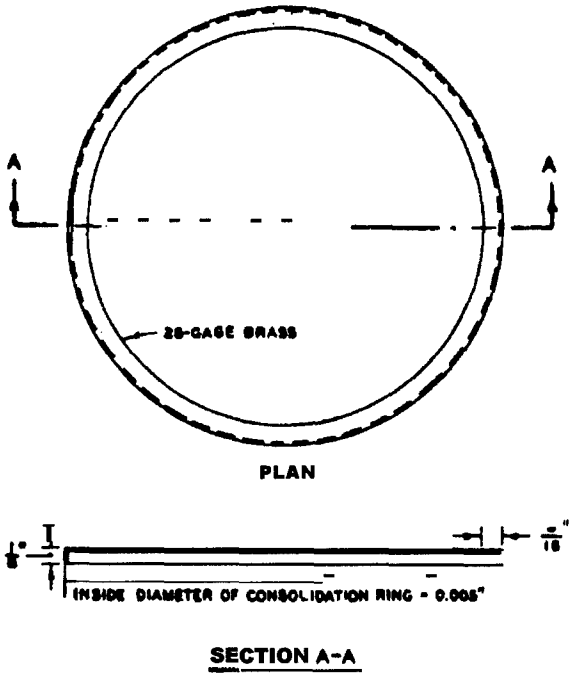


Figure 3. Typical retainer ring

d. Loading devices of various types may be used to apply load to the specimen. The most commonly used is the beam-and-weight mechanism. The loading device should be capable of transmitting axial load to the specimen quickly and gently. Also, the equipment should be capable of maintaining the load constant for at least 24 hr. The equipment should be calibrated to ensure that the loads indicated are those actually applied to the soil specimen.

e. Dial Indicator. A dial indicator reading counterclockwise, with a range of 0.50 in. and graduated to 0.0001 in., is recommended.

f. Equipment for Preparing Specimens. A trimming turntable operated as a vertical lathe is commonly used in preparing specimens (see Fig. 4). Suitable trimming knives notched to fit the thickness of the consolidation ring, a wire saw with 0.01 -in. -diameter wire, and a metal straightedge or screed are also required.

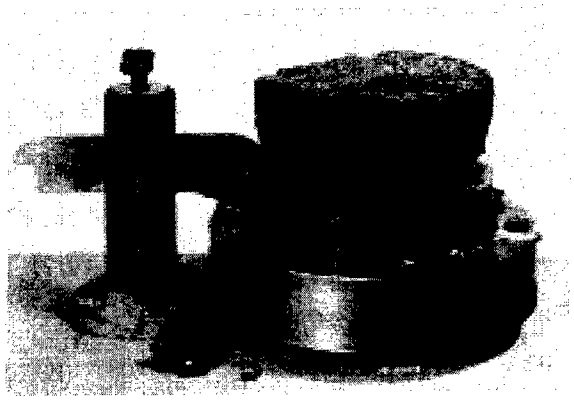


Figure 4. Cutting. specimen into specimen ring

g. Other items needed are:

(1) Balances, 'sensitive to 0.1 g and 0.01 g

(2) Timing device, a watch or clock with second hand

(3) Centigrade thermometer,, range 0 to 50 C, accurate to 0.1 c

(4) Distilled or demineralized water

(5) Filter papers and glass plates

(6) Apparatus necessary to determine water content and specific gravity (see Appendices I, WATER CONTENT - GENERAL, and IV, SPECIFIC GRAVITY)

3. CALIBRATION OF EQUIPMENT. In the consolidation test it is desired to measure only the volume change of the specimen; therefore, corrections must be applied for any significant deformation due to the compressibility of the apparatus itself. In sandy and stiff soils, an appreciable proportion of the total deformation may be caused by this factor. Therefore, a calibration curve should be prepared for each consolidometer when testing such soils. This is done by placing the consolidometer with submerged porous stones and filter papers in the loading device, applying the load increments to be used in the consolidation test, and reading the dial

indicator for each load. After the maximum load has been applied, the loads are decreased in the same order as that in which they were applied, and the dial indicator reading is again recorded. Since the deformations are almost instantaneous, the effect of time can be ignored. The total change in dial reading for each load is the correction to be applied to the dial reading recorded during the consolidation test under that same load. Generally, a single cycle will be sufficient for the calibration.

4. PREPARATION OF SPECIMENS. Specimens shall be prepared in a humid room to prevent evaporation of soil moisture. Extreme care shall be taken in preparing specimens of sensitive soils to prevent disturbance of their natural structure. Specimens of relatively soft soils may be prepared by progressive trimming in front of a calibrated, ring-shaped specimen cutter as shown in Figure 2 of Appendix II, UNIT WEIGHTS, VOID RATIO, POROSITY, AND DEGREE OF SATURATION. More commonly, specimens are prepared using the trimming turntable shown in Figure 4 herein; the procedure, based on the use of this equipment, shall be as described in the following subparagraphs. Preferably, specimens of compacted soil should be compacted to the desired density and water content directly into the consolidation ring, in thin (1/4 to 3/8 in.) layers, using a pressing or kneading action of a tamper having an area less than one-sixth the area of the specimen and thoroughly scarifying the surface of each layer before placing the next. Alternatively, specimens may be trimmed from samples compacted in a compaction mold by a similar kneading action.

a. Using a wire saw, knives, or other tools, trim the specimen into approximately cylindrical shape with a diameter about 1/2 in. greater than the inside diameter of the specimen ring. Care should be taken to disturb the specimen as little as possible during trimming. Chamfer the lower edge of the specimen until the bottom will fit exactly into the specimen ring.

b. Place the specimen ring on the rotating wheel and the specimen on the ring, starting the bottom into the ring as shown in Figure 4. Use a cutting tool to trim the specimen to accurate dimensions, place a glass

plate on top of the specimen, and gently force the specimen down during the trimming operation. The specimen should fit snug in the consolidation ring.

c. Cut off the portion of the specimen remaining above the ring with a wire saw or knife (or other convenient tool for harder specimens). Extreme care must be taken for many soils, especially fissured clays, in cutting off this portion. Carefully true the surface flush with the specimen ring with a straightedge. If a pebble is encountered in the surface, remove it and fill the void with soil. Place a glass plate (previously weighed) over the ring and turn the specimen over.* Cut off the soil extending beyond the bottom of the ring in the same manner as that described for the surface portion. Place another glass plate on this surface, and again invert the specimen to an upright position, removing the metal disk if one was used.

5. PROCEDURE. The procedure shall consist of the following steps:

a. Record all identifying information for the specimen, such as project number, boring number, and other pertinent data, on the data sheet (Plate VIII-1 is a suggested form); note any difficulties encountered in preparation of the specimen. Measure and record the height and cross-sectional area of the specimen. Record weight of specimen ring and glass plates. After specimen is prepared, record the weight of the specimen plus tare (ring and glass plates), and from the soil trimmings obtain 200 g of material for specific gravity‡ and water content determinations. Record

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* It may be found convenient after the top surface has been prepared to place over the specimen a circular metal plate, approximately 0.05 in. thick and of the same diameter as the specimen, and force it down until it is flush with the top of the ring. This provides a recess for the top porous stone and prevents the specimen from squeezing out of the consolidation ring.

‡ It is recommended that a specific gravity test be made on representative material from every consolidation test specimen.

the wet weight of the material used for the water content determination on the data sheet.

b. Fill the grooves in the base of the consolidometer with water. Fit the porous stone (previously saturated with water) into the base of the consolidometer. Add sufficient water so that the water level is at the top of the porous stone. Place a moist filter paper (Whatman No. 1 or equal) over the porous stone. (Be very careful to avoid entrapping any air during the assembly operations .) Place the ring with the specimen therein on top of the porous stone. If the fixed-ring consolidometer is used, secure the ring to the base by means of clamps and screws.

c. Place a moist filter paper on top of the specimen, and then place the previously saturated top porous stone and the loading plate in position.

d. Place the consolidometer containing the specimen in the loading device.

e. Attach the dial indicator support to the consolidometer, and adjust it so that the stem of the dial indicator is centered with respect to the specimen. Adjust the dial indicator to permit the approximate maximum travel of the gage but still allow measurement of any swelling.

f. Adjust the loading device until it just makes contact with the specimen. The seating load should not exceed about 0.01 ton per sq ft.

g. Read the dial indicator, and record the reading on a data sheet (Plate VIII-2 is a suggested form). This is the initial reading of the dial indicator.

h With the specimen assembled in the loading device, apply a load of 0.25 ton per sq ft to the specimen and immediately inundate the specimen by filling the volume within the inundation ring or the chamber surrounding the specimen with water. If a fixed-ring device is used, a low head of water should be applied to the base of the specimen and maintained during the test by means of the standpipe. Place a thermometer in the water, and record the temperature at 2-hr intervals.

To obtain reliable time-consolidation curves the temperature should not vary more than ± 2 C during the test. For most fine-grained soils a load of 0.25 ton per sq ft is usually enough to prevent swelling, but if swelling occurs apply additional load increments until swelling ceases. Were the specimen permitted to swell, the resulting void ratio-pressure curve would have a more gradual curvature and the preconsolidation pressure would not be well defined. Alternatively to applying a large initial load increment, swelling can be prevented by not inundating the specimen until the load on the specimen has reached such a level that consolidation is obviously occurring along the straight-line portion of the void ratio-pressure curve, During the stages before water is added, the humidity around the specimen should be maintained at 100 percent to prevent evaporation; a moist paper towel, cotton batting, or other cellular material wrapped around the specimen is usually adequate. This alternative procedure permits an initial load increment less than 0.25 ton per sq ft to be applied to the specimen.

i. Continue consolidation of the specimen by applying the next load increment. The following loading schedule is considered satisfactory for routine tests: 0.25, 0.5, 1.0, 2.0, 4.0, 8.0, and 16.0 tons per sq ft, the total load being doubled by each load increment. The maximum load should be great enough to establish the straight-line portion of the void ratio-pressure curve, subsequently described. The loading schedule may be modified by the designer to simulate the loading sequence anticipated in the field.

j. Observe and record on the data sheet (Plate VIII-2) the deformation as determined from dial indicator readings after various elapsed times. Readings at 0.1, 0.2, 0.5, 1.0, 2.0, 4.0, 8.0, 15.0, and 30.0 min, and 1, 2, 4, 8, and 24 hr for each load increment are usually satisfactory. A timing device should be located near the consolidometer to insure accurately timed measurements. Allow each load increment to remain on the specimen for a minimum of 24 hr until it is determined that the primary

consolidation is completed. For most plastic, fine-grained soils a time interval of 24 hr will be sufficient. It is desirable that the duration of all load increments be the same. During the course of the test, plot the dial reading versus time data for each load increment on a semilogarithmic plot as shown in Plate VIII-3. Plot the dial reading on an arithmetic scale (ordinate) and the corresponding elapsed time on a logarithmic scale (abscissa) as shown in Figure 5. For saturated fine -grained soils, the dial reading versus time curve will generally be similar to the curve shown in Figure 5 and can be converted into a time-consolidation curve using the theory of consolidation. The 100 percent consolidation or the completion of the primary consolidation is arbitrarily defined as the intersection of the tangent to the curve at the point of inflection, with the tangent to the straight-line portion representing the secondary time effect. The construction necessary for determination of the coordinates

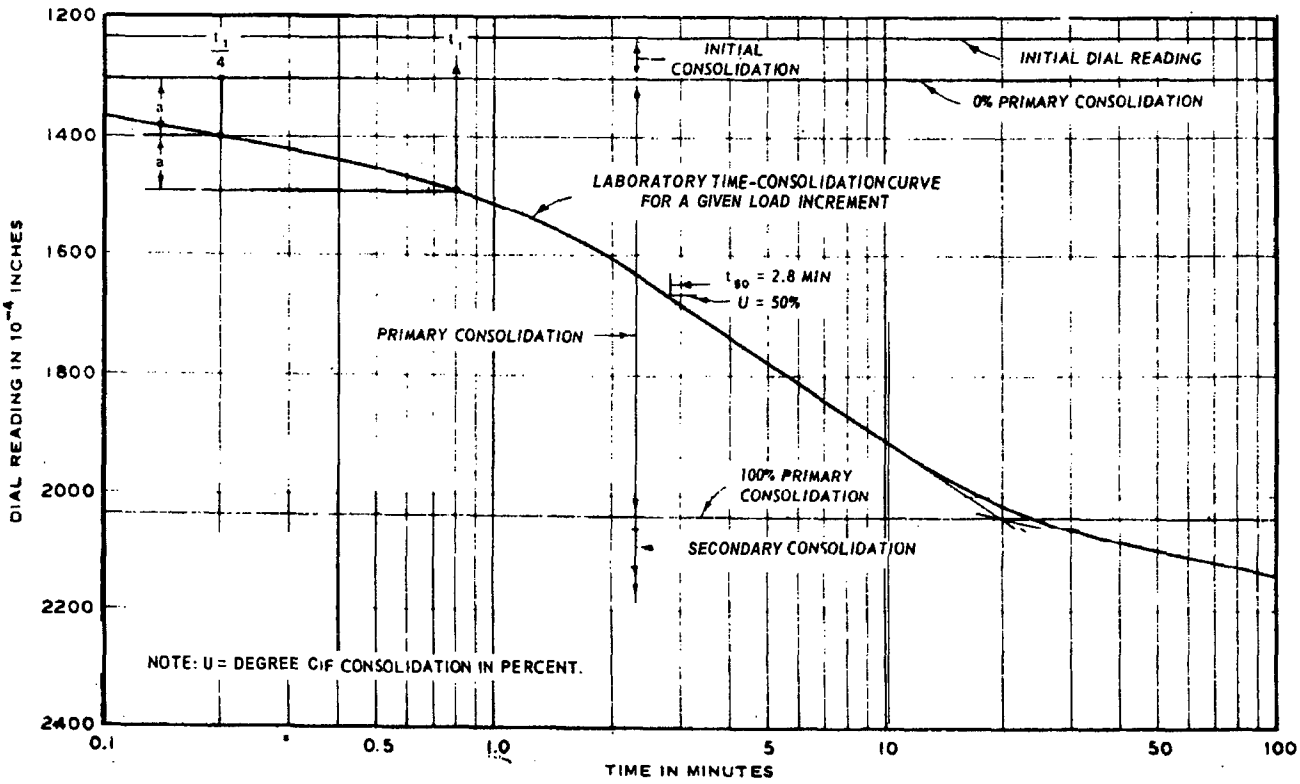


Figure 5. Time -consolidation curve

representing 100 percent consolidation and other degrees of consolidation is shown in Figure 5 (see also EM 1110-2-1 904, Settlement Analysis).

k. Record on a data sheet (Plate VIII-4 is a suggested form) the dial reading for each load increment corresponding to a selected time (usually 24 hr) at which primary consolidation has been completed for all increments.

l. After the specimen has consolidated under the maximum load, remove the load in decrements, taking three-quarters of the load off successively for each of the first two decrements and as considered desirable thereafter. Take readings of the dial indicator as each decrement is removed to determine the rebound of the specimen. Observe, record, and plot the dial readings versus time; loads should not be removed until the dial readings are relatively constant with time or until the dial reading versus logarithm of time curve indicates completion of rebound. The final load at the end of the rebound cycle should be 0.1 ton per sq ft or less, and this load should be maintained for 24 hr in order to reduce to a tolerable amount the error in the final water content determination caused by swelling.

m. When the dial readings indicate no further significant rebound, remove the dial indicator and disassemble the apparatus. Carefully blot any excess water from the ring and surface of the specimen, eject the specimen into a dish of known weight, and weigh the dish and wet specimen; then oven-dry the wet specimen to constant weight (see Appendix I, WATER CONTENT - GENERAL).

6. COMPUTATIONS. The computations shall consist of the following:

a. From the recorded data compute and record on the data sheet, Plate VIII-l, the initial and final water contents (see Appendix I, WATER CONTENT - GENERAL). Compute also the height of solids, void ratio before and after test, initial and final degree of saturation, and dry density

before test using the following formulas:†

$$\text{Height of solids, } H_s = \frac{W_s}{A \times G_s \times \gamma_w} = \left[\frac{W_s}{A \times G_s \times 1 \times 2.54} \text{ in.}\right]$$

$$\text{Void ratio before test, } e_o = \frac{H - H_s}{H_s}$$

$$\text{Void ratio after test, } e_f = \frac{H_f - H_s}{H_s}$$

$$\text{Initial degree of saturation, percent, } S_o = \frac{H_{wo}}{H - H_s} \times 100$$

$$\text{Final degree of saturation, percent, } S_f = \frac{H_{wf}}{H_f - H_s} \times 100$$

$$\text{Dry density before test, } \gamma_d = \frac{W_s}{H \times A} = \left[\frac{W_s \times 62.4}{H \times A \times 2.54} \text{ lb per cu ft}\right]$$

where

W_s = weight of dry soil, g

A = area of specimen, sq cm

G_s = specific gravity of solids

γ_w = unit weight of water, g per cc

H = height of specimen, in.

H_f = height of specimen at end of test = $H - \Delta H$, in., where ΔH is the net change in height of specimen

† Equations in brackets are based on units of measurements shown in explanation of symbols.

$$H_{wo} = \text{original height of water} = \frac{W_{wo}}{A \times \gamma_w} = \left[\frac{W_{wo}}{A \times 1 \times 2.54} \text{ in.}\right] \text{ where}$$

W_{wo} = weight of water in specimen before test, g

$$H_{wf} = \text{final height of water} = \frac{W_{wf}}{A \times \gamma_w} = \left[\frac{W_{wf}}{A \times 1 \times 2.54} \text{ in.}\right] \text{ where}$$

W_{wf} = weight of water in specimen after test, g

The purpose of computing the degree of saturation at the beginning and end of the test is to obtain a check on the accuracy of the data observed and recorded. An appreciable variation from 100 percent in the computed degree of saturation at the beginning of the test for specimens that are known to be completely saturated may indicate the presence of gas or air in the specimen, or an error in the data or computations.

b. From data sheet (Plate VIII-2) or from dial reading-time plots (Plate VIII-3), obtain the final dial reading for each load increment that corresponds to the selected time interval (usually 24 hr) and record these values on the data sheet (Plate VIII-4). On the same data sheet record the dial reading correction. The dial reading correction is the dial reading corresponding to the deformation of the apparatus for each load, and is obtained from a calibration curve for the apparatus as described in paragraph 3. The height of voids, H_v, corresponding to any given load is equal to the initial height of voids ($H - H_s$) minus the corrected dial reading (ΔH). The change in height of the specimen is equal to the accumulative change of the corrected dial readings. Compute the void ratios of the specimen corresponding to different load increments. The void ratio is numerically equal to the height of the voids divided by the height of solids.

7. PRESENTATION OF RESULTS. The results of the consolidation

tests shall be shown on the report forms, Plates VIII-3 and VIII-5. The data shall be shown graphically in terms of time-consolidation curves in the form shown as Plate VIII -3 and in terms of void ratio -pressure curves in the form shown as Plate VIII-5. To obtain the void ratio-pressure curve, the void ratio, e, is plotted on the arithmetic scale (ordinate) and the corresponding pressure, p, in tons per square foot on the logarithmic scale (abscissa) as shown in Figure 6. The overburden pressure, p_o, preconsolidation pressure, p_c, and compression index, C_c, shall be determined and shown on the report form (Plate VIII-5). The determinations of the preconsolidation and overburden pres - sures of a soil are normally made by design engineers; procedures are

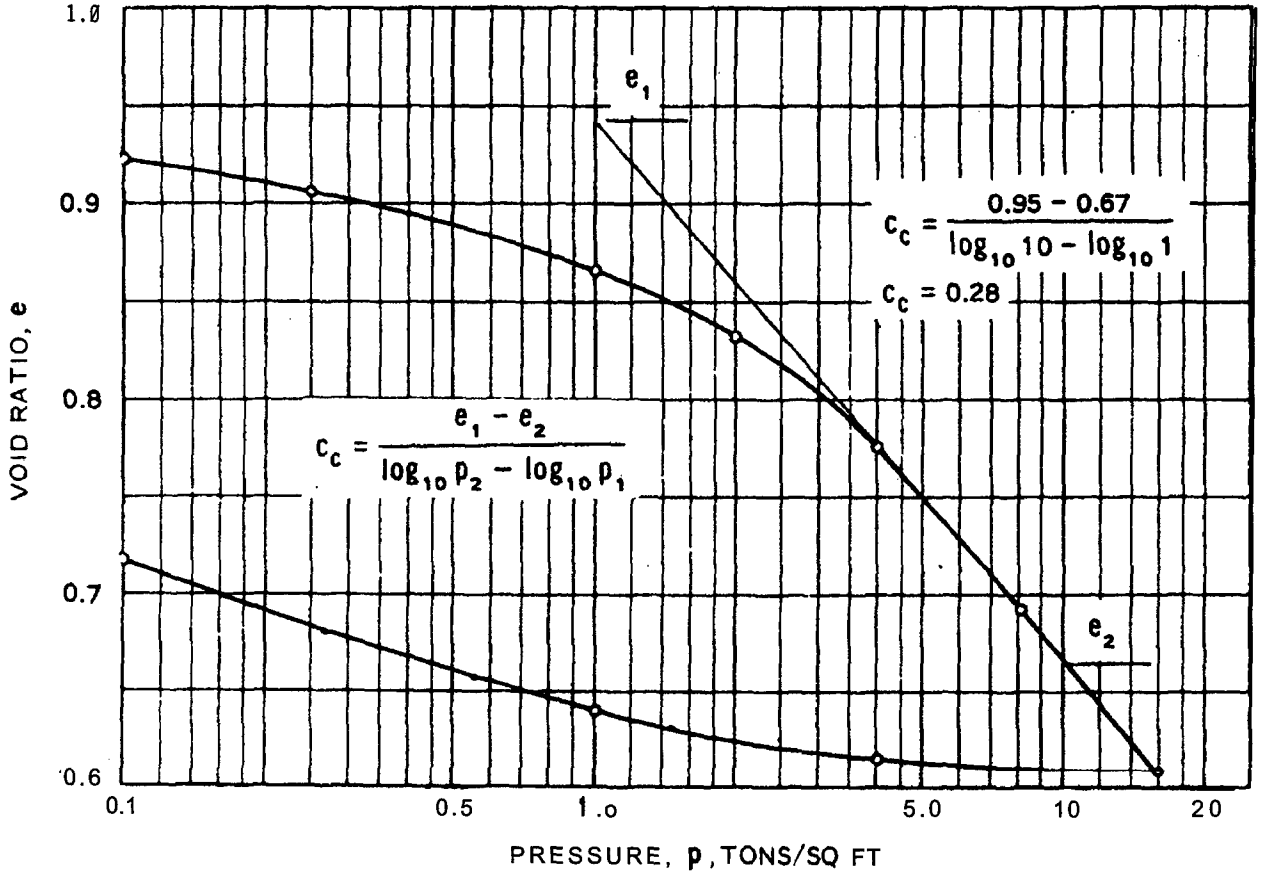


Figure 6. Void ratio-pressure curve

discussed in detail in EM 1110-2-1904, Settlement Analysis.

The slope of the straight-line portion of the pressure-void ratio curve on a semilogarithmic diagram is known as the compression index, C_c. The compression index is defined by the equation

$$C_c = \frac{e_1 - e_2}{\log_{10} P2 - \log_{10} p_1}$$

where p_1 and p_2 are selected pressures from the straight-line portion of the curve, and e_1 and e_2 are the corresponding void ratios. The compression index is a measure of the compressibility of a soil. An example of the computation of C_c is shown in Figure 6. For simplification, p_2 is often chosen to be 10 times p_1, in which case the denominator becomes unity.

If permeability tests are performed in conjunction with the consolidation test (see Appendix VII, PERMEABILITY TESTS), the coefficient of permeability for each load increment shall also be plotted in the form shown as Plate VIII-5.

A brief description of undisturbed specimens should be given on the report form. The description should include color, approximate consistency, and any unusual features (such as stratification, fissures, shells, roots, sand pockets, etc.). For compacted specimens, give the method of compaction used and the relation to maximum density and optimum water content.

8. POSSIBLE ERRORS. Following are possible errors that would cause inaccurate determinations of consolidation characteristics:

a. Specimen disturbance during trimming. As in all laboratory determinations of the engineering properties of undisturbed soils, the largest errors are caused by changing the natural structure of the soil while preparing the test specimen; disturbance will affect the time-deformation relation and will obscure the preconsolidation pressure.

Trimming must be done in the humid room with every care taken to minimize the disturbance. Since the zone of disturbance caused by trimming a given soil is essentially constant in depth, the effect of this zone can be reduced by using a larger specimen.

b. Specimen not completely filling ring. The volume of the specimen must be exactly that of the consolidation ring, otherwise there will not be complete lateral confinement.

c. Galvanic action in consolidometer. To prevent changes in the consolidation characteristics of the specimen due to galvanic currents, all metal parts of the consolidometer should be of the same noncorrosive material; it is preferable that all such parts be made of plastic.

d. Permeability of porous stones too low. The measured rate of consolidation can be markedly affected by the permeability of the porous stones. The stones should be cleaned after every test to remove embedded soil particles.

e. Friction between specimen and consolidation ring. Tests have shown that over 20 percent of the load applied to a specimen can be lost by side friction in a fixed-ring consolidometer and about one-half this amount in a floating-ring consolidometer.† The effect of side friction can be reduced by (1) using a larger diameter specimen, (2) using a thinner specimen, and (3) lining the consolidation ring with Teflon.

f. Inappropriate load increment factor. Depending on the purpose of the test, a load increment factor of 2.0 (that is, of doubling the total load by each load increment) may not be satisfactory; for example, to sharply define the preconsolidation pressure, a factor of 1.50 or even 1.25 may be better.

g. Unsatisfactory height (or thickness) of specimen. The height of the specimen will determine how clearly can be detected the break in

† D. W. Taylor, Research on the Consolidation of Clays, Serial 82, Department of Civil and Sanitary Engineering, Massachusetts Institute of Technology (Cambridge, Mass., 1942).

the time-consolidation curve that represents completion of primary consolidation. Depending on the character of the soil, if the specimen is too thin, the time to 100 percent consolidation may be too rapid, while if too thick, the break in the curve may be obscured by secondary compression. Also, when a load increment factor smaller than 2.0 is used, the thickness of the specimen may have to be increased to cause enough deformation during primary consolidation to define the break in the curve.

CONSOLIDATION TEST

(Specimen Data)

Date _____

Project _____

Boring No. _____ Sample No. _____

Classification

			Before Test			After Test	
			Specimen		Trimmings	Specimen	
Tare No.			Ring and plates				
Weight in grams	Tare plus wet soil						
	Tare plus dry soil						
	Water	W_w	W_{wo}			W_{wf}	
	Tare						
	Dry soil	W_s					
	Water content	w	w_o	%	%	w_f	%

Consolidometer No.		Area of specimen, A, sq cm	
Weight of ring, g		Height of specimen, H, in.	
Weight of plates, g		Sp gr of solids, G_s	

Height of solids, $H_s = \frac{W_s}{A \times G_s \times \gamma_w} = \frac{\quad}{\quad \times \quad} \times 1 \times 2.54 =$ in.

Original height of water, $H_{wo} = \frac{W_{wo}}{A \times \gamma_w} = \frac{\quad}{\quad} \times 1 \times 2.54 =$ in.

Final height of water, $H_{wf} = \frac{W_{wf}}{A \times \gamma_w} = \frac{\quad}{\quad} \times 1 \times 2.54 =$ in.

Net change in height of specimen at end of test, $\Delta H =$ in.

Height of specimen at end of test, $H_f = H - \Delta H =$ in.

Void ratio before test, $e_o = \frac{H - H_s}{H_s} = \frac{\quad}{\quad} =$

Void ratio after test, $e_f = \frac{H_f - H_s}{H_s} = \frac{\quad}{\quad} =$

Degree of saturation before test, $S_o = \frac{H_{wo}}{H - H_s} = \frac{\quad}{\quad} =$ %

Degree of saturation after test, $S_f = \frac{H_{wf}}{H_f - H_s} = \frac{\quad}{\quad} =$ %

Dry density before test, $\gamma_d = \frac{W_s}{H \times A} = \frac{\quad \times 62.4}{\quad \times 2.54} =$ lb/cu ft

Remarks _____

_____

Technician _____ Computed by _____ Checked by _____

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PLATE VIII-1

CONSOLIDATION TEST

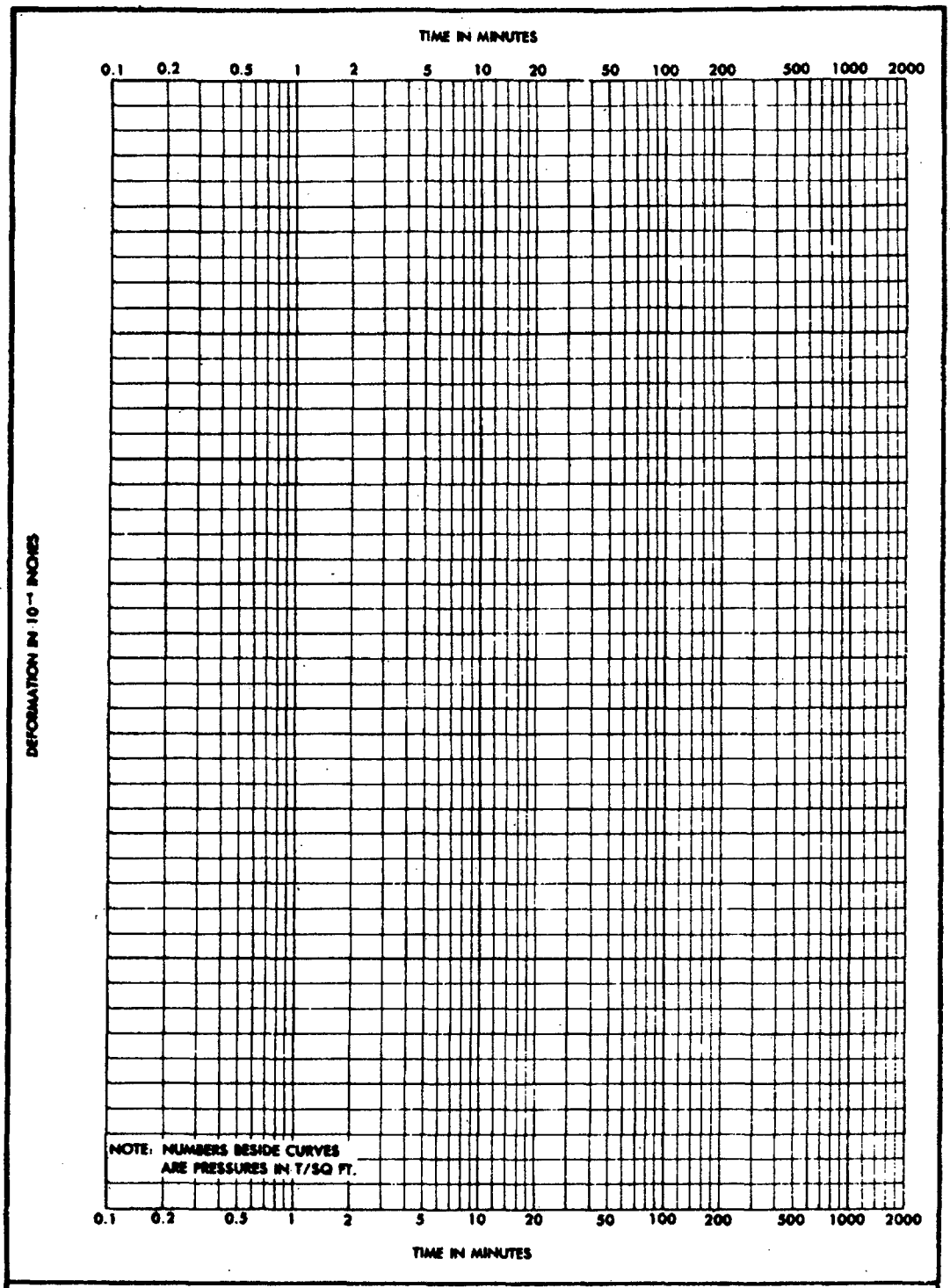
(Time-Consolidation Data)

Date _____

Project _____

Boring No. _____ Sample No. _____ Consol. No. _____

Date and Pressure	Time	Elapsed Time, min	Dial Rdg. 10^{-4} in.	Temp. °C	Date and Pressure	Time	Elapsed Time, min	Dial Rdg. 10^{-4} in.	Temp. °C



PROJECT			
AREA			
BORING NO.	SAMPLE NO.	DEPTH EL	DATE

ENG FORM 2088 1 MAY 63 PREVIOUS EDITIONS ARE OBSOLETE. **CONSOLIDATION TEST—TIME CURVES** (TRANSLUCENT)

PLATE VIII-3

CONSOLIDATION TEST

(Computation of Void Ratios)

PROJECT _____ DATE _____

BORING NO. _____ SAMPLE NO. _____ CONSOLIDOMETER NO. _____

Pressure, p T/sq ft	Date Increment Applied	Time in Min Increment Effective	Dial Reading 10^{-4} in.	Correction 10^{-4} in.	Change in Height, ΔH 10^{-4} in.	Height of Voids, H_v 10^{-4} in.	Void Ratio, e

Note:

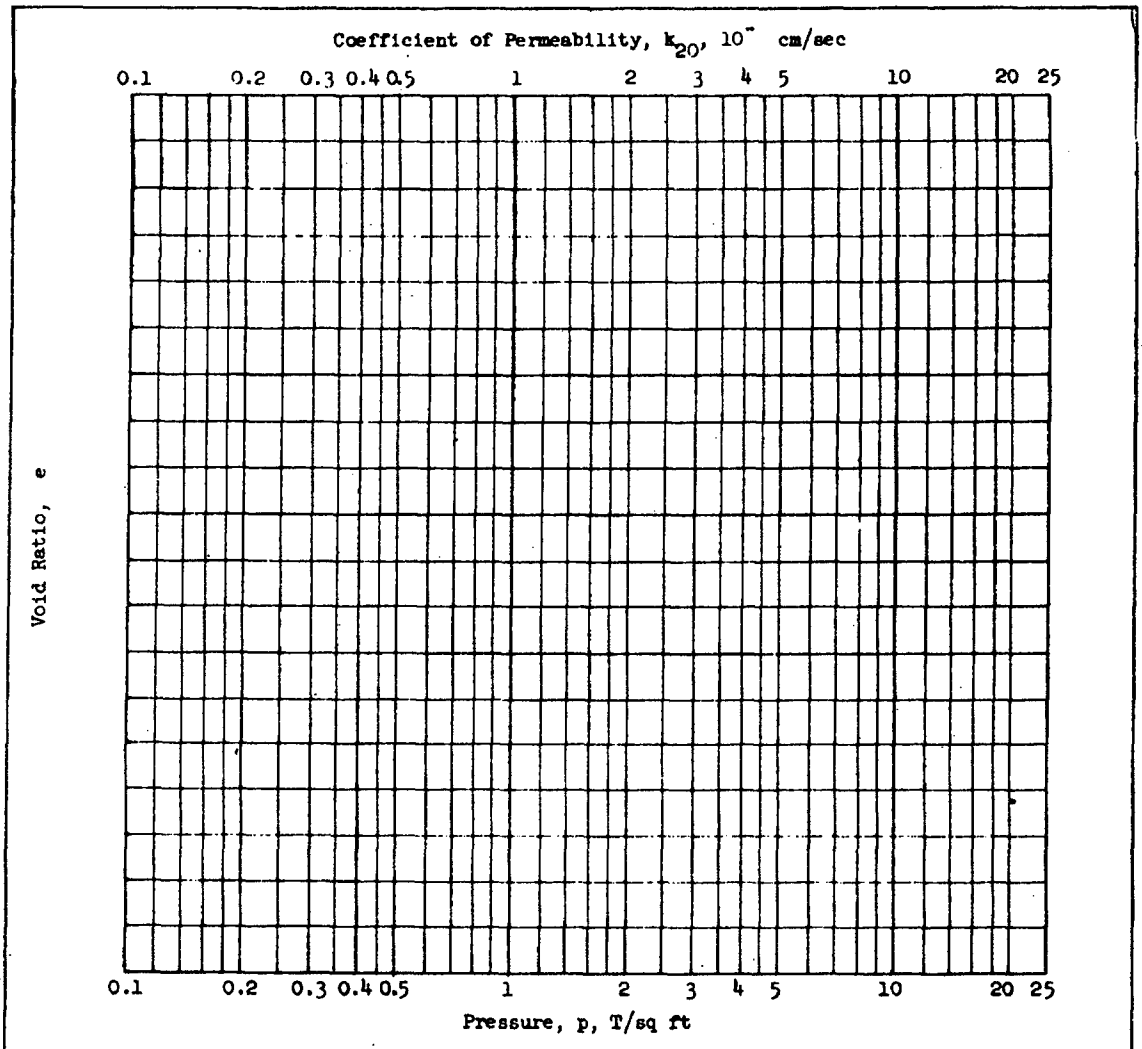
Height of voids, $H_v = (H - H_s) - \Delta H =$

Void ratio, $e = \frac{H_v}{H_s}$

TECHNICIAN _____ COMPUTED BY _____ CHECKED BY _____

ENG FORM 3849
1 JUN 65

PLATE VIII-4



Type of Specimen		Before Test		After Test	
Diam in.	Ht in.	Water Content, w_o	%	w_f	%
Overburden Pressure, p_o	T/sq ft	Void Ratio, e_o		e_f	
Preconsol. Pressure, p_c	T/sq ft	Saturation, S_o	%	S_f	%
Compression Index, C_c		Dry Density, γ_d	lb/ft^3		
Classification		k_{20} at e_o =	$\times 10^{-}$ cm/sec		
LL	G_s	Project			
PL	D_{10}				
Remarks		Area			
		Boring No.	Sample No.		
		Depth El	Date		
		CONSOLIDATION TEST REPORT			

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PLATE VIII-5

APPENDIX VIIIA:

SWELL AND SWELL PRESSURE TESTS

1. INTRODUCTION. Swell is the process of imbibing available moisture to cause an increase in soil volume until the pore water pressure increases to an equilibrium determined by the environment. The amount of swell to satisfy the new pore pressure equilibrium depends on the magnitude of the vertical loading and soil properties that include soil composition, natural water content and density, and soil structure. The rate of swell depends on the coefficient of permeability (hydraulic conductivity), thickness, and soil properties. Soils that are more likely to swell appreciably include clays and clay shales with plasticity indices greater than 25, liquid limits greater than 40, and natural water contents near the plastic limit or less.

The presence of capillary stress or negative pore water pressure arising from molecular forces in swelling soils causes available moisture to be absorbed. The vertical confining pressure required to prevent volume expansion from absorbed moisture is defined as the swell pressure.

The swell and swell pressure are generally determined in the laboratory with the one-dimensional consolidometer (Appendix VIII, CONSOLIDATION TEST). Swell is determined by subjecting the laterally confined soil specimen to a constant vertical pressure and by giving both the top and bottom of the specimen access to free water (usually distilled) to cause swell. The swell pressure is determined by subjecting the laterally confined soil specimen to increasing vertical pressures, following inundation, to prevent swell.

2. APPARATUS, CALIBRATION OF EQUIPMENT, AND PREPARATION OF SPECIMENS. The apparatus is essentially the same as that listed in paragraph 2, Appendix VIII, CONSOLIDATION TEST.

Smoothly ground porous stones should be used in the consolidometer to minimize seating displacements. Filter paper should not be used because it is compressible and contributes to displacements. The equipment is calibrated and the sample prepared in the same manner as described for the consolidation test.

3. PROCEDURE.

a. Swell Test. (1) Record all identifying information for the specimen, such as project number, boring number, and other pertinent data, on the data sheet (Plate VIII-J, p VIII-18, is a suggested form); note any difficulties encountered in preparation of the specimen. Measure and record the height and cross-sectional area of the specimen. Record weight of specimen ring and glass plates. After the specimen is prepared, record the weight of the specimen plus tare (ring and glass plates), and from the soil trimmings obtain 200 g of material for specific gravity and water content determinations. Record the weight of the material used for the water content determination on the data sheet.

(2) Fit an air-dried, smoothly ground porous stone into the base of a dry consolidometer. Place the ring with the specimen therein on top of the porous stone. If the fixed-ring consolidometer is used, secure the ring to the base by means of clamps and screws.

(3) Place the top air-dried, smoothly ground porous stone and loading plate in position. The inside of the reservoir should be moistened to promote a high-humidity environment. The reservoir and loading plate should subsequently be covered with a sheet of impervious material such as plastic film or moist paper towel to inhibit loss of moisture.

(4) Place the consolidometer containing the specimen in the loading device.

(5) Attach the dial indicator support to the consolidometer, and adjust it so that the stem of the dial indicator is centered with

respect to the specimen. Adjust the dial indicator to allow for both swell and consolidation measurements.

(6) Adjust the loading device until it just makes contact with the specimen. The seating load should not exceed about 0.01 ton per sq ft.

(7) Read the dial indicator and record the reading on a data sheet (Plate VIII-2, p VIII-19, is a suggested form). This is the initial reading of the dial indicator.

(8) Depending on the particular design considerations, a specific load (e.g. overburden plus design load) is applied to the specimen. After a period of at least 5 min but less than 30 min (to avoid shrinkage from drying), record the dial reading on the data sheet (Plate VIII-2) and inundate the specimen.

(9) Inundate the specimen by filling the reservoir, within the inundation ring that surrounds the specimen, with water (distilled, tap, or field pore water, actual or reconstituted). Cover with the plastic film, and moist paper towel or equivalent. If a fixed-ring device is used, a low head of water should be applied to the base of the specimen and maintained during the test by means of the sandpipe.

(10) Observe and record on the data sheet (Plate VIII-2) the deformation as determined from dial indicator readings after various elapsed times. Readings at 0.1, 0.2, 0.5, 1.0, 2.0, 4.0, 8.0, 15.0, and 30.0 min, and 1, 2, 4, 8, 24, 48, and 72 hr are usually satisfactory. A timing device should be located near the consolidometer to ensure accurately timed measurements. Allow the load increment to remain on the specimen until it is determined that the primary swell is completed. Time to complete the primary swell of heavy clay soils and clay shales often requires three or more days. Plot the dial reading versus time data on a semilogarithmic plot as shown in Plate VIIIA-1 to ascertain the completion of primary swell. The completion of

primary swell is arbitrarily defined as the intersection of the tangent to the curve at the point of inflection, with the tangent to the straight-line portion representing a secondary time effect as shown in Figure 1. This is similar to the procedure in paragraph 5j, Appendix VIII, CONSOLIDATION TEST.

(11) Although a falling-head permeability test may be performed at this point of the swell test (see paragraph 8, Appendix VII, PERMEABILITY TESTS), the specimen may not be fully saturated and the permeability results consequently affected. After primary swell is complete,† the load should be removed in decrements according to the procedure in paragraphs 5l and 5m, Appendix VIII, CONSOLIDATION TEST. The final load should be the seating load.

b. Swell Pressure Test. The procedure of this test is identical with the preceding swell test through (9). Following (9), increments of load are applied as needed to prevent swell. Variations from the dial reading at the time the specimen is inundated with water should be kept preferably within 0.0002 in. and not more than 0.0005 in. Following 24 hr after the specimen exhibits no further tendency to swell, a falling-head permeability test may be performed (see paragraph 8, Appendix VII, PERMEABILITY TESTS) and the final load (which is also the swell pressure) should be removed in decrements according to the procedure in paragraphs 5l and 5m, Appendix VIII, CONSOLIDATION TEST. The final load should be the seating load.

4. COMPUTATIONS. The computations for the swell tests are similar to those presented in paragraph 6, Appendix VIII, CONSOLIDATION TEST.

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† After primary swell is complete, the loading pressure may be increased to consolidate the specimen until the void ratio is less than the initial void ratio under the overburden pressure; thereafter, loads may subsequently be removed to determine rebound characteristics. This procedure permits an alternative measure of the swell pressure, defined as the total pressure required to reduce the void ratio to the initial void ratio.

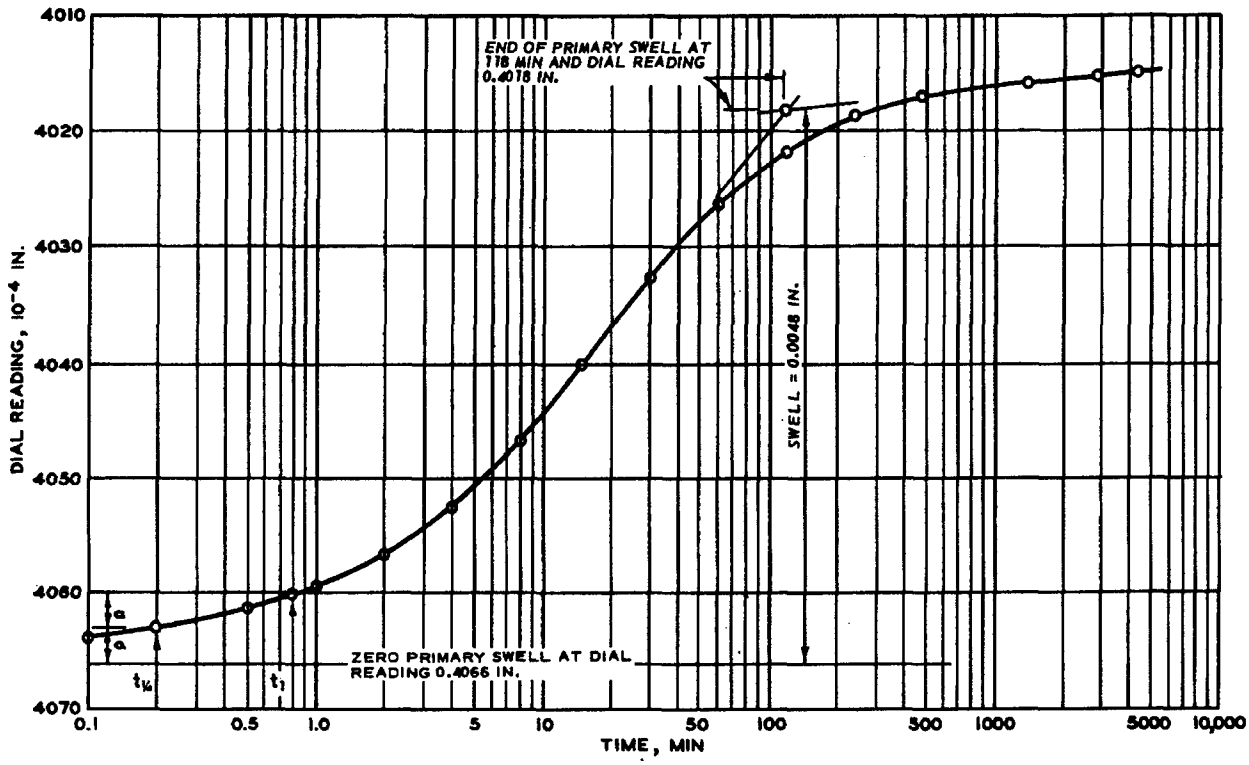


Figure 1. Time-swell curve

5. PRESENTATION OF RESULTS. The results of the swell tests shall be summarized on report forms, Plates VIIIA-1 and VIIIA-2. The data shall be shown graphically in terms of time-swell curves on the form shown as Plate VIIIA-1 and in terms of void ratio-pressure curves on the form shown as Plate VIIIA-2. To obtain the void ratio-pressure curve, the void ratio, e, is plotted on the arithmetic scale (ordinate) and the corresponding pressure, p, in tons per square foot on the logarithmic scale (abscissa) as shown in Figures 2 and 3. The overburden pressure, p_o, swell pressure, p_s, swell index, C_s, and swell at p_o, $\Delta H/H$, shall be determined and shown on the report form (Plate VIIIA-2). The determination of the overburden pressure is normally made by design engineers.

The swell pressure, p_s, is determined as in Figure 3. The swell pressure by the alternative definition in the footnote of paragraph 3

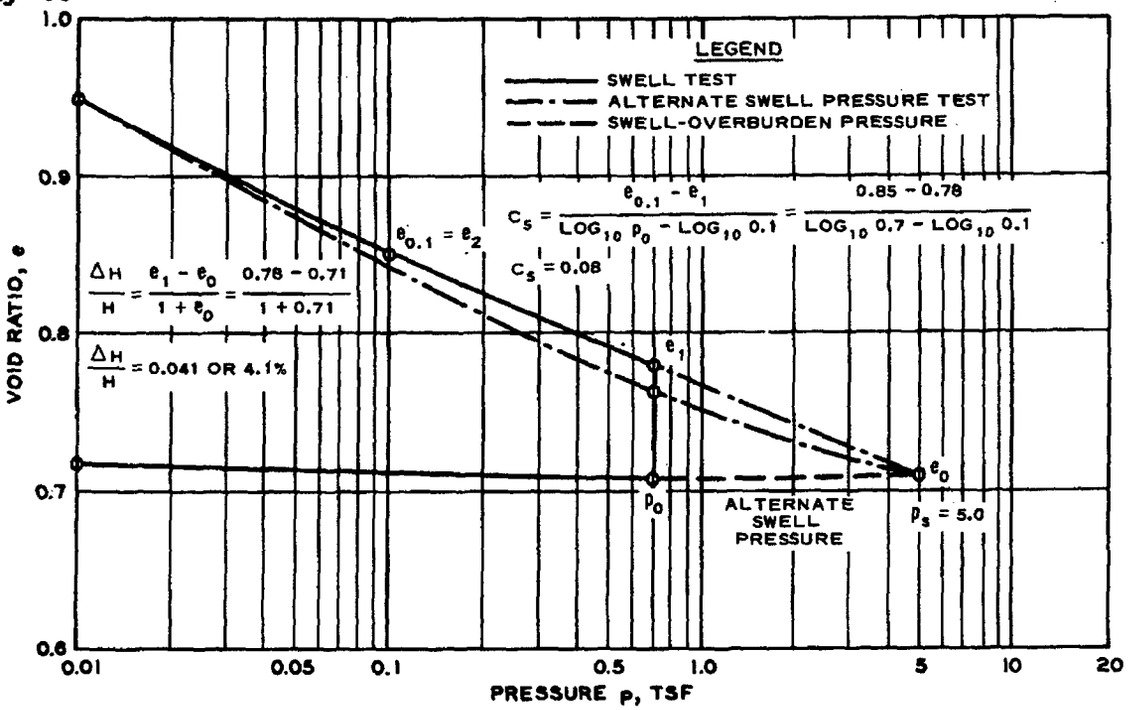


Figure 2. Void ratio-pressure curve of swell test

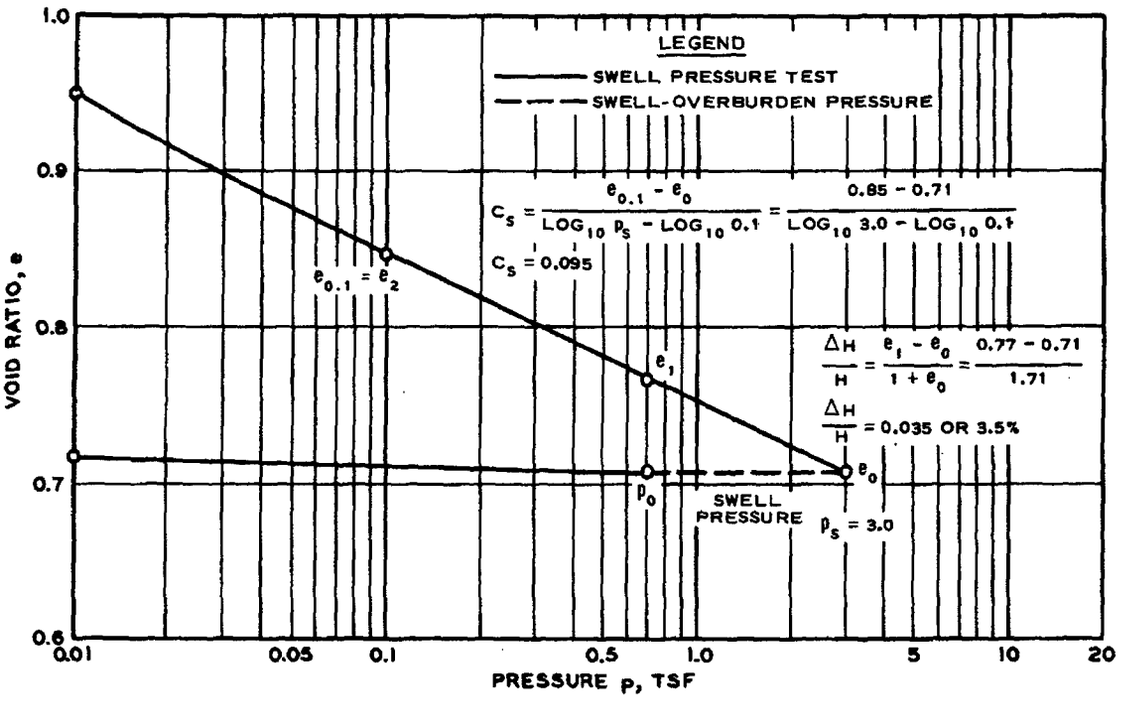


Figure 3. Void ratio-pressure curve of swell pressure test

may also be determined from the results of the swell test (Fig. 2) and recorded on Plate VIIIA-2. The initial void ratio is the void ratio determined following placement of the overburden pressure.

The swell index is defined by the equation

$$C_s = \frac{e_2 - e_1}{\log_{10} p_1 - \log_{10} p_2}$$

where p_1 and p_2 are selected pressures from a straight-line portion of a void ratio-pressure rebound curve, and e_1 and e_2 are the corresponding void ratios. The swell index is a measure of the ability of the soil to swell. Example computations of C_s are shown in Figures 2 and 3.

The swell is defined by the equation

$$\frac{\Delta H}{H} = \frac{e_1 - e_0}{1 + e_0}$$

where e_1 is the void ratio following swell, and e_0 is the void ratio prior to swell. Example computations are shown in Figures 2 and 3 for swell at the overburden pressure p_o.

If permeability tests are performed in conjunction with the swell tests (see Appendix VII, PERMEABILITY TESTS), the coefficient of permeability determined for each void ratio during rebound shall also be plotted in the form shown as Plate VIIIA-2.

A brief description of undisturbed specimens should be given on the report form. The description should include color, approximate consistency, and any unusual features (such as stratification, fissures, shells, roots, sand pockets, etc.). For compacted specimens, give the method of compaction used and the relation to maximum density

and optimum water content.

6. POSSIBLE ERRORS. In addition to the possible errors discussed in paragraph 8, Appendix VIII, CONSOLIDATION TEST, the following may also cause inaccurate determination of swelling characteristics:

a. Displacements caused by seating of the specimen against the surface of the porous stones may be significant, especially if swell displacements and loading pressures are small. Thus, smoothly ground porous stones are recommended.

b. Filter paper is highly compressible and contributes to the observed displacements and hysteresis in displacements on loading and rebound. Filter paper is consequently not recommended.

c. The compressibility characteristics of the consolidometer and the test procedures influence the swell pressure results. Because very small expansions in volume greatly relieve swell pressures, the stiffness of the consolidometer should be as large as possible, and variations in displacements that occur during determination of the swell pressure should be as small as possible.

d. Swelling characteristics determined by consolidometer swell tests for the purpose of predicting heave of foundation and compacted soils are not representations of many field conditions because:

(1) Lateral swell and lateral confining pressures are not simulated.

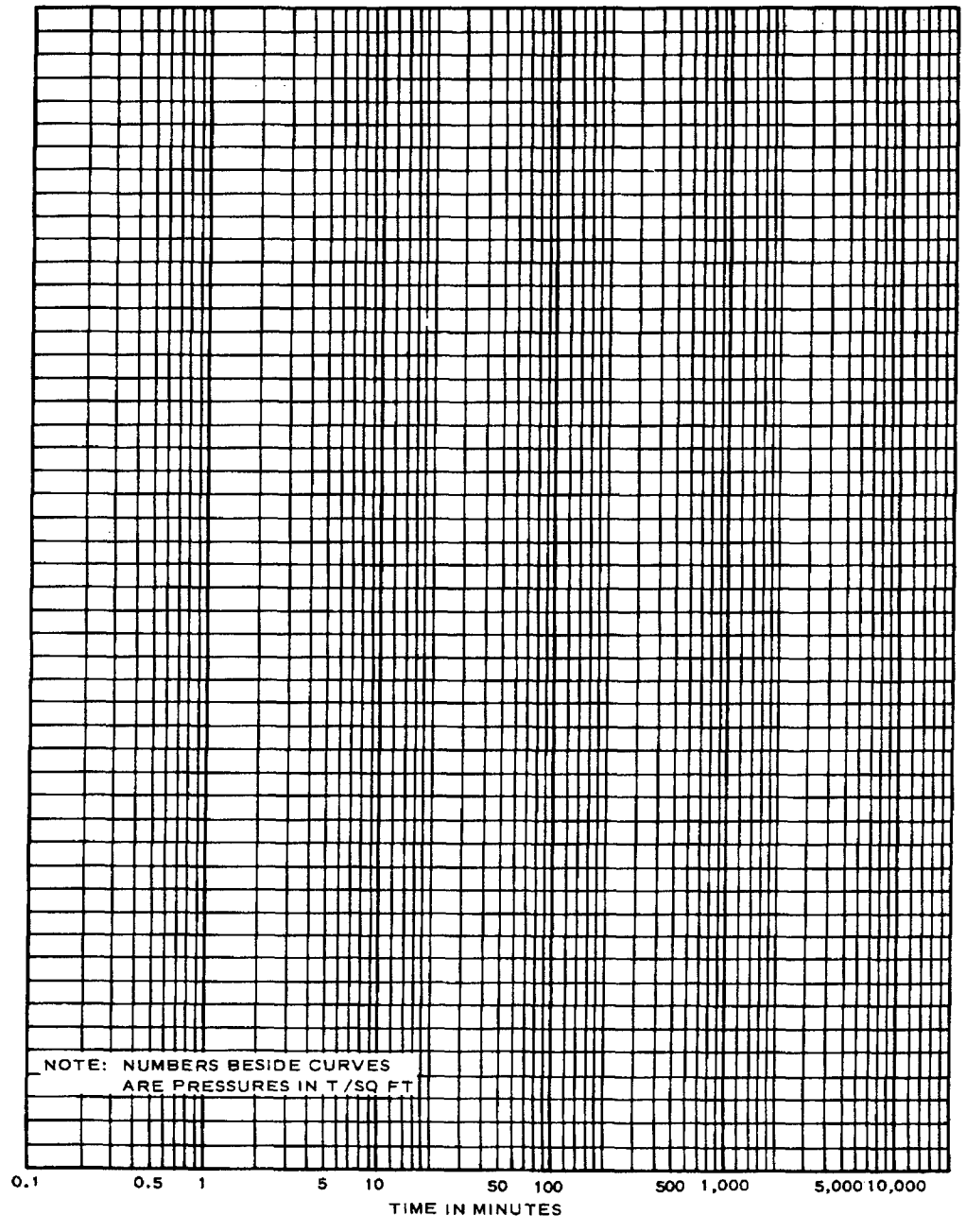
(2) The actual availability of water to the foundation soils may be cyclic or intermittent. Field swell usually occurs under constant overburden pressure depending on the availability of water. The swell index, in contrast, is evaluated by observing swell due to decreases in overburden pressure while the soil specimen is inundated with water.

(3) Rates of swell indicated by swell tests are not reliable indicators of field rates of swell due to fissures in the mass soil and inadequate simulation of the actual availability of water to the soil.

(4) Secondary or long-term swell, which is not evaluated by these test procedures, may be significant for some clays and clay shales . These soils may not be fully saturated at the conclusion of the swell test.

(5) Chemical content of the inundating water affects results; e.g., when testing shales, distilled water may give radically different results than natural or reconstituted pore water.

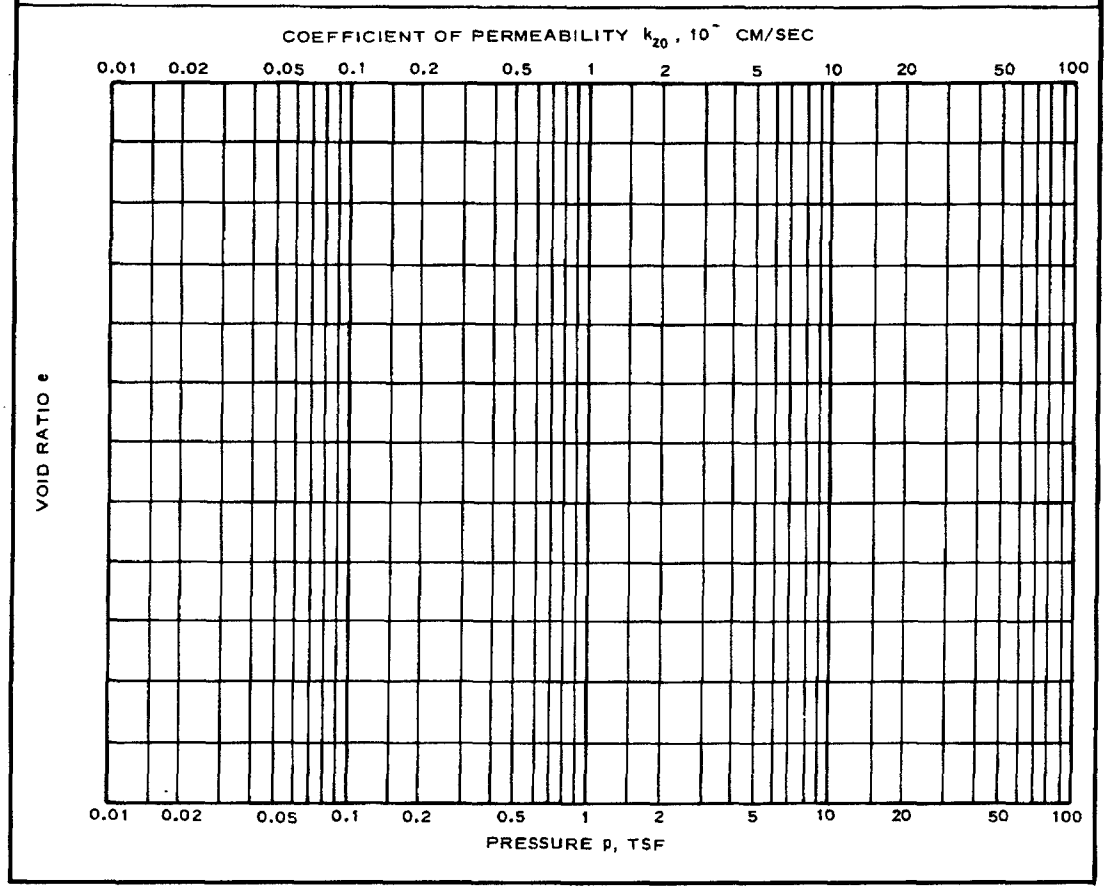
SWELL TEST-TIME CURVES (TRANSLUCENT)		Date
Project		
Boring No.	Sample No.	Depth El



ENG FORM 1 MAY 78 4663 –R

PLATE VIIIA-1

SWELL TEST REPORT			Date		
Project					
Boring No.	Sample No.	Depth El			
Type of Specimen		Type of Swell Test			
Diam in.	Ht in.	Before Test	After Test		
Overburden Pressure, P_o TSF		Water Content, w_o	w_o		
Swell Pressure, p_s TSF		Void Ratio, e_o at p_o %	e_f		
Swell Index, C_s		Saturation, S_o %	S_f		
Swell at p_o, $\frac{\Delta H}{H}$ %		Dry Density, γ_d lb/ft^3			
Classification		k_{20} at p_o = x 10^{-} cm/sec			
LL	G_s				
PL	D_{10}				
Remarks					



ENG FORM 1 APR 80 4664 –R

PLATE VIIIA-2

APPENDIX IX:

DRAINED (S) DIRECT SHEAR TEST

1. INTRODUCTION. The direct shear test is used to measure the shear strength of a soil under drained conditions. In this test, a relatively thin, square specimen of soil is placed in a rigid box that is divided horizontally into two frames, the specimen is confined under a vertical or normal stress, and a horizontal force is applied so as to fail the specimen along a horizontal plane at its midheight. Generally, a minimum of three specimens, each under a different normal stress, are tested to establish the relation between shear strength and normal stress. The magnitude of the normal stresses used depends on the range of stresses anticipated for design. Because of the difficulties involved in controlling drainage of the soil specimen during the direct shear test, only the drained (S) test method, in which complete consolidation is permitted under each increment of normal and shear stress, shall be used.

2. APPARATUS. The apparatus should consist of the following:

a. Shear box, of bronze or stainless steel, open at the top and divided horizontally into two frames that can be fitted together accurately with alignment pins and elevating screws. A schematic diagram of a direct shear box is shown in Figure 1. The lower frame of the

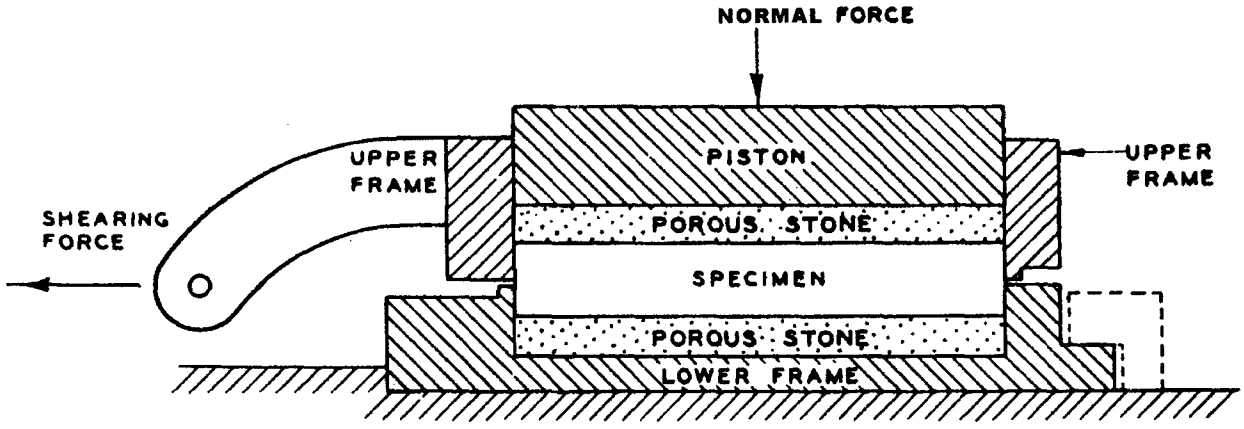


Figure 1. Schematic diagram of direct shear box

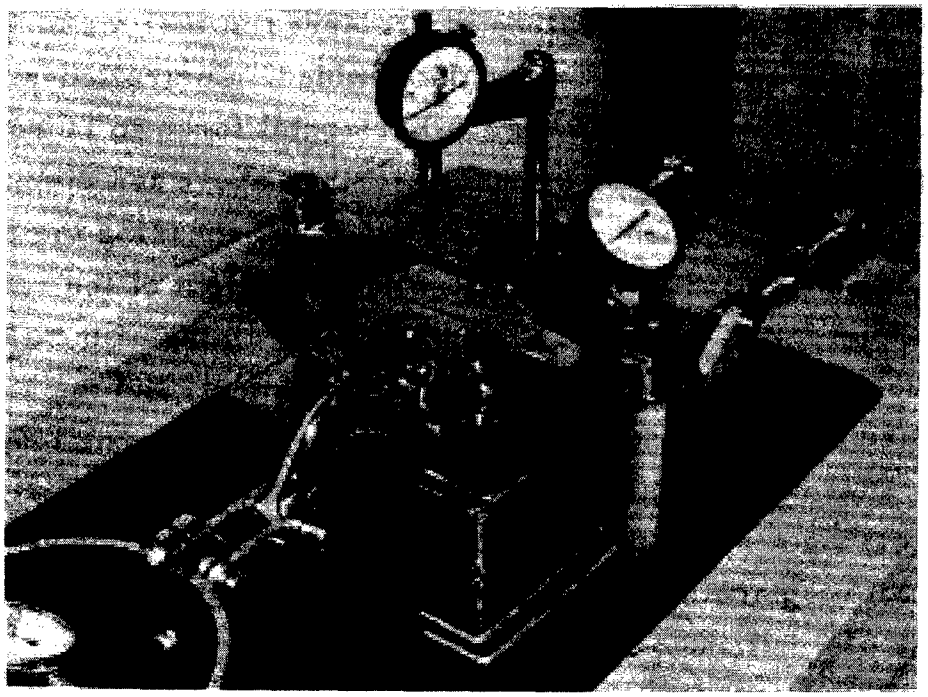
shear box shall contain a reservoir for water, with the bottom grooved or provided with a grooved area to permit drainage. The upper frame of the shear box should contain an accurately machined piston, the bottom of which is also grooved to permit drainage. The upper frame shall be provided with horizontal locking screws to lock it to the piston within the upper frame of the box. The various metal parts of the shear box shall be of a like, noncorrosive material. A typical shear box, assembled and unassembled, is shown in Figure 2; a narrow-edged shear box as shown is preferable to a wide-edged one. Generally, shear boxes for direct shear tests shall have minimum inside dimensions of 3 by 3 in. The maximum thickness of a 3- by 3-in. specimen shall be 1/2 in. after consolidation. If the soil to be tested contains particles larger than the No. 4 sieve, the test should be performed in a larger shear box or else the shear strength should be determined by means of the triaxial test.

b. Porous stones shall be smooth, coarse grade Alundum or Carborundum, finish-ground except for the surface in contact with the specimen, which shall be rough-finished by sandblasting or by using hand-tools. Porous metal plates of similar porosity and texture may also be used. It is very important that the permeability of porous stones not be reduced by the collection of soil particles in the pores of the stones; hence, frequent checking and cleaning (by flushing and boiling, or by ultrasonic cleaner) are required to ensure the necessary permeability (see page VIII-3).

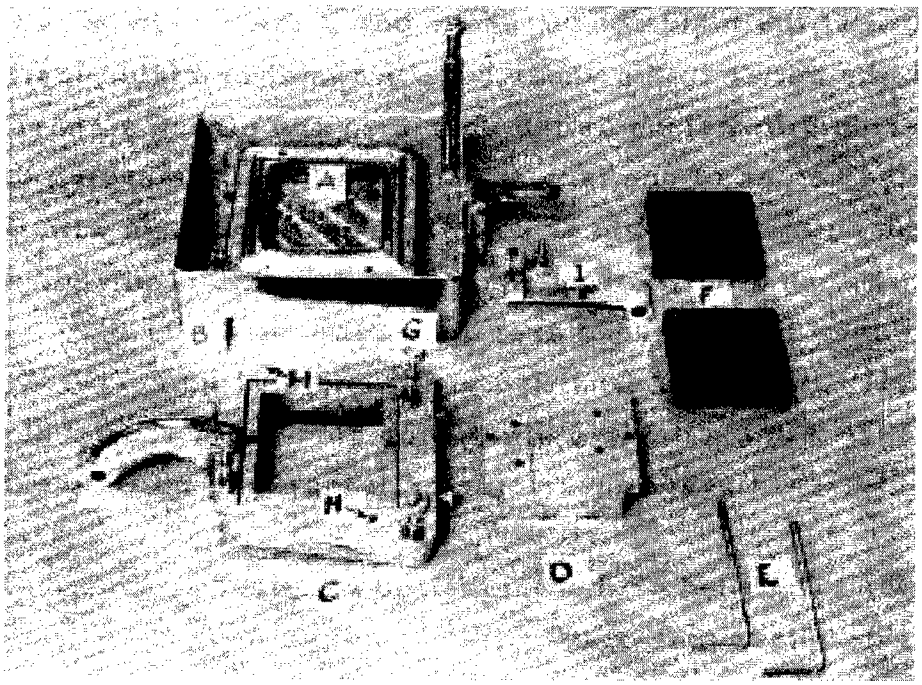
c. Loading devices for applying the normal load and horizontal shear force to the specimen. Any type of loading device may be used that meets the following requirements:

(1) For applying normal load. The equipment for applying the normal load shall be capable of transmitting the load to the specimen quickly, without impact, and maintaining the load constant for the duration of the test. The equipment should be calibrated to ensure that the loads indicated are those actually applied to the soil specimen.

(2) For applying shear force. The horizontal shear force may be applied by either controlled-stress or controlled-strain methods,



a. Assembled in loading apparatus



b. Unassembled

A, lower frame; B, water reservoir; C, upper frame, D, piston; E, alignment pins; F, porous stones; G, elevating screws; H, holes for alignment pins; I, bracket for vertical dial indicator.

Figure 2. Typical direct shear box

though the controlled-strain method is preferred in that the ultimate, as well as the maximum, stress can be determined. Controlled-stress equipment should be capable of applying the horizontal force in increments to the specimen in the same manner as that described above for the normal load. Controlled-strain equipment should be capable of shearing the specimen at a uniform rate of strain and should permit adjustment of the rate of strain over a relatively wide range. The controlled straining of the specimen is usually done with a motor and gear box arrangement, and the shear force is determined by a load-indicating device such as a proving ring or frame.

d. Dial indicators for measuring (1) vertical deformation of the specimen, having a range of 0.25 in. and an accuracy of 0.0001 in.; and (2) horizontal displacement of the specimen, having a range of 0.5 in. and an accuracy of 0.001 in.

e. Equipment for preparing specimen including a specimen cutter with sharp cutting edges. The cutter shall have inside dimensions the same as those of the inside of the shear box if the specimen is to be transferred from the cutter to the shear box. A metal or rigid plastic plate having the same dimensions as the specimen and having a short handle attached at the center of one face is required for transferring the specimen from the cutter to the shear box. Knives, wire saws with 0.010-in.-diameter wire, and other cutting equipment are also required.

f. Other items needed are:

(1) Balances, sensitive to 0.1 and 0.01 g.

(2) Timing device, a watch or clock with second hand.

(3) Centigrade thermometer, range 0 to 50 C, accurate to 0.1 C.

(4) Distilled or demineralized water.

(5) Glass plates.

(6) Apparatus necessary to determine water content and specific gravity (see Appendixes I, WATER CONTENT - GENERAL, and IV, SPECIFIC GRAVITY).

3. PREPARATION OF SPECIMEN. A sample sufficient to provide a minimum of three identical specimens is required. Specimens shall be prepared jn a humjd room to prevent evaporation of moisture. The specimen is generally prepared by progressive trimming in front Of the specimen cutter. However, satisfactory specimens of hard soils often may be obtained by cutting and trimming wjthout using the specimen cutter. Specimens of very soft, sensitive soils may be obtained more conveniently by pushing the cutter into the sample without preliminary trimming. Extreme care shall be taken jn preparjng undisturbed specimens of sensitive soils to prevent disturbance of their natural structure. Preferably, specimens of compacted soil should be trimmed from samples compacted in a compaction mold, using a pressing or kneading action of a tamper having an area less than one-sixth the area of the sample. A less desirable procedure is to compact the soil to the desired density and water content directly in the shear box, in a single layer using a similar kneading action. The procedure usually used in preparing specimens by progressive trimming follows :

a. Cut a sample of soil approximately 1-1/4 in. high and 4-1/2 in. in diameter from the sample to be tested.

b. Place the sample of soil on a glass plate and center the specimen cutter on top of the sample. Push the cutter vertically into the sample not more than 1/4 in. and carefully trjm the soil from the edge of the Cutter. Repeat the operation until the specimen protrudes above the top of the cutter. Care should be taken to insure that no voids are formed between the cutter and specimen.

C. Remove the portion of the specimen protruding above the cutter, using a wire saw for soft specimens, and a straightedge, knives, or other convenient tools for harder specimens. Trim the specimen flush with the top of the cutter. If a pebble or other protrusion is encountered on the surface, remove it and fill the void with soil.

d. Place a previously weighed glass plate on the surface of the

specimen. Many soils will adhere to glass; consequently, it is advisable to use waxed paper or similar material between the specimen and glass plate. Invert the specimen, trim the bottom as described in step c, and on this surface place another weighed glass plate.

e. From the soil trimmings obtain 200 g of material for water content and specific gravity determinations (see Appendixes I, WATER CONTENT - #GENERAL, and IV, SPECIFIC GRAVITY).

f. Repeat the procedures outlined above to produce two additional specimens.

4. PROCEDURE. a. Preliminary. The procedure for setting up the test specimen shall consist of the following:

(1) Record all identifying information for the specimen, such as project, boring number, and other pertinent data, on the data sheet (Plate IX-1 is a suggested form); note any difficulties encountered in preparation of the specimen. Measure the inside area and height of the shear box and record as the initial dimensions of the specimen on the data sheet. Weigh and record the weight of specimen plus tare (specimen cutter and glass plates).

(2) Assemble the shear box with the upper frame held in alignment with the lower frame by means of the alignment pins or screws. Place a previously saturated porous stone, rough side up, on the baseplate of the shear box in the bottom of the lower frame.

(3) Insert the specimen cutter, sharpened edge first, into the upper frame of the shear box until it is wedged firmly and is parallel with the top of the upper frame (the inside edge of the upper frame should be beveled slightly to accept the cutter). Lay a piece of waxed paper, slightly smaller than the specimen, on the surface of the specimen, and with a smooth, continuous press of the transferring plate, force the specimen from the cutter and into firm contact with the porous stone. While pressing the specimen from the cutter, care must be exercised to prevent tilting or otherwise disturbing the specimen. Withdraw the transferring

plate, remove the specimen cutter, and peel the waxed paper from the specimen. Place a previously saturated porous stone, rough side down, on top of the specimen and lower the piston onto this porous stone.

(4) Place the shear box in position on the loading apparatus. At this stage of the test the upper and lower frames are in contact. Assemble the loading equipment, and mount the two dial indicators to be used for measuring vertical and horizontal deformation. The dial indicator measuring vertical deformation should be set so that it can measure movement in either direction.

b Consolidation. The procedure for consolidating the specimen shall consist of the following:

(1) Apply the desired normal load gently to the specimen. The range of normal loads for the three specimens will depend on the loadings expected in the field. The maximum normal load should be at least equal to the maximum normal load expected in the field in order that the shear strength data need not be extrapolated for use in design analysis. Generally, normal loads less than about 3 tons per sq ft may be applied in a single increment, whereas greater normal loads should be applied in several increments to prevent the soil from squeezing out of the box, For very soft soils it is usually necessary to apply even the relatively lighter normal loads in increments.

(2) As soon as possible after applying the normal load, fill the water reservoir with distilled or demineralized water to a point above the top of the specimen. Maintain this water level during the consolidation and subsequent shear phases so that the specimen is at all times effectively submerged.

(3) Allow the specimen to drain and consolidate under the desired normal load or increments thereof prior to shearing. During the consolidation process, record on the data sheet (see Plate VIII-2 of Appendix VIII, CONSOLIDATION TEST) the vertical dial readings after various elapsed times. Readings at 0.1, 0.2, 0.5, 1, 2, 4, 8, 15, and 30 min,

and 1, 2, 4, 8, and 24 hr for each increment of normal load are usually satisfactory. During the course of the test, plot the dial readings versus elapsed time on a semilogarithmic plot (see Fig. 5 of Appendix VIII, CONSOLIDATION TEST). Allow each load increment to remain on the specimen until it is determined that primary consolidation is complete (see Appendix VIII, CONSOLIDATION TEST).

c. Shear Test. The procedure for shearing the specimen after consolidation shall consist of the following:

(1) Raise the upper frame of the shear box about 1/16 in. by turning the elevating screws. The amount of clearance between the upper and lower frames should be sufficient to prevent the two frames from coming in contact during the shear test, yet not permit the soil to extrude between the frames. Lock the upper frame to the loading piston by means of the horizontal locking screws. In raising the upper frame, the applied load on the specimen is increased by an amount equal to the weight of the upper frame. Adjust vertical load by reducing applied load by this amount. Retract the elevating screws.

(2) Remove the alignment pins.

(3) Shear the specimen at a relatively slow rate so that a fully drained condition (no excess pore pressures) exists at failure. The following equation shall be used as a guide in determining the minimum time required from start of test to shear failure:

$$t_f = 50t_{50}$$

where

t_f = total. elapsed time to failure in minutes

t_{50} = time in minutes required for the specimen to achieve 50 percent

consolidation† under the normal load or increments thereof (see Fig. 5 of Appendix VIII, CONSOLIDATION TEST). It is to be noted, however, that time-consolidation curves indicated by soils that exhibit a tendency to swell under a given increment of normal load are not meaningful and, therefore, cannot be used in determining minimum times required to failure. In such instances, the following procedures may be used to obtain valid time-consolidation curves:

(a) An increment of normal load is applied to the specimen and the specimen is inundated with water and allowed to come to equilibrium; the time-consolidation curve for any increment of normal stress applied thereafter is valid.

(b) Alternatively, the specimen may be inundated with water following the completion of primary consolidation under the final increment of normal load. However, the specimen must be allowed to come to equilibrium after inundation (prior to shear). Prior to inundation, the specimen should be maintained in a humid atmosphere by covering the shear box and filling the water reservoir with moist paper towels, cotton, or other cellular material.

(4) Considerable experience and judgment are generally required in determining the proper rate of shear load application. The following discussions will provide guidance in this respect:

(a) Controlled-stress test. The rate of load application may be determined approximately by the following procedure. Estimate the maximum shear stress and select an initial load increment of about 10 percent of the estimated failure load. Apply each increment to the specimen and permit at least 95 percent consolidation before applying the next increment. The time-consolidation curve obtained during the

---

† If the time for 50 percent consolidation is difficult to determine, values for higher percentages of consolidation may be used to compute t_f. The following relations may be used: $t_f = 35t_{60} = 25t_{70} = 18t_{80} = 12t_{90}$.

consolidation sequence of the test may be used for determining if 95 percent consolidation has been achieved. When 50 to 70 percent of the estimated failure load has been applied to the specimen (depending on the shape of the stress-deformation curve), reduce the size of the increments to one-half the initial size. As failure is approached, use a series of increments equal to one-fourth of the initial load increment to accurately define the failure load.

(b) Controlled-strain test. The rate of strain may be determined approximately by dividing the estimated horizontal deformation at maximum shear stress by the computed time to failure, t_f. The test shall be continued until the shear stress becomes essentially constant, as shown in Figure 3b, or until a horizontal deformation of 0.5 in. has been reached.

(5) The horizontal and vertical deformations and the applied shear force shall be observed at convenient intervals. Plate IX-2 is a

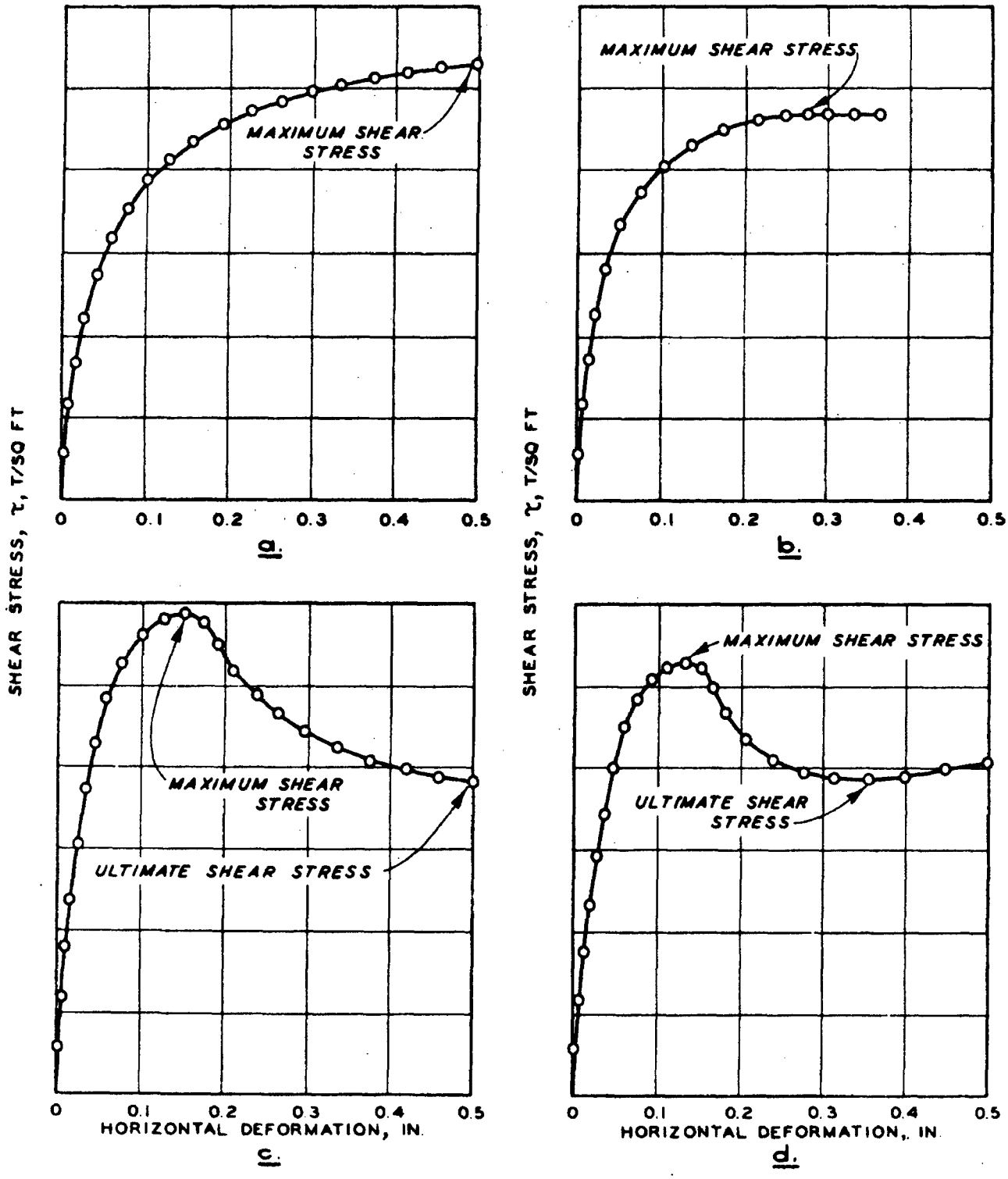


Figure 3. Examples of stress-deformation curves

suggested form for recording the observations.

(6) Remove the specimen from the shear box, blot any excess moisture, and trim away a minimum of 1/16 in. from all sides of the specimen to form a rectangular block. Determine the water content of the specimen (see Appendix I, WATER CONTENT - GENERAL). The dry weight of the specimen should be computed using the water content based on specimen trimmings taken at the start of the test.

5. COMPUTATIONS. The computations shall consist of the following:

a. From the recorded data compute and record on the data sheet (see Plate IX-1) the initial and final water contents (see Appendix I, WATER CONTENT - GENERAL). Compute also the void ratio before test, after consolidation, and after test, and the initial and final degrees of saturation, and dry density before test, using the following equations:†

$$\text{Void ratio before test, } e_o = \frac{V_o - V_s}{V_s}$$

$$\text{Void ratio after consolidation, } e_c = \frac{V_c - V_s}{V_s}$$

$$\text{Void ratio after test, } e_f = \frac{V_f - V_s}{V_s}$$

† Equations in brackets are based on units of measurement shown on the following page in explanation of symbols.

$$\text{Initial degree of saturation, } S_o, \text{ percent} = \frac{\frac{w_o}{100} \times \frac{W_s}{\gamma_w}}{V_o - V_s} \times 100,$$

$$\text{or } \left[\frac{w_o \times W_s}{V_o - V_s}\right]$$

$$\text{Final degree of saturation, } S_f, \text{ percent} = \frac{\frac{w_f}{100} \times \frac{W_s}{\gamma_w}}{V_f - V_s} \times 100,$$

$$\text{or } \left[\frac{w_f \times W_s}{V_f - V_s}\right]$$

$$\text{Dry density before test, } \gamma_d, \text{ lb per cu ft} = \left[\frac{W_s}{H_o \times A} \times 62.4\right]$$

where

W_s = weight of dry soil, g
A = area of specimen, sq cm
G_s = specific gravity of solids
γ_w = unit weight of water, g per cc
w_o = water content of specimen before test, percent
w_f = water content of specimen after test, percent
H_o = initial height of specimen, cm
H_c = height of specimen after consolidation, cm
H_f = height of specimen at end of test, cm
V_s = volume of solids, cc; W_s/G_s
V_o = volume of specimen before test, cc; $A \times H_o$
V_c = volume of specimen after consolidation, cc; $A \times H_c$
V_f = volume of specimen after test, cc = $A \times H_f$

Units of measurement are those commonly used in computations for the direct shear tests.

b. Complete the data sheet, Plate IX-2.

c. The shear stress, τ, in tons per square foot may be calculated from the following equation:

$$\tau = \frac{F}{A} \times 0.465$$

where

F = applied shear force, lb

A = horizontal cross-sectional area of the specimen, sq cm (assumed to be constant and equal to the initial area for routine testing)

6. PRESENTATION OF RESULTS. The results of the direct shear test shall be shown on the report form, Plate IX-3. The shear stress and vertical deformation during shear shall be plotted versus the horizontal deformation. As shown in Figure 3, the maximum shear stress, τ_{max}, is either the actual maximum or peak shear stress or, if the shear stress increases continuously during the test, the shear stress at 0.5-in. horizontal deformation. When the shear stress decreases after reaching a maximum value, the minimum shear stress attained before 0.5-in. horizontal deformation is considered to be the ultimate shear stress, τ_{ult}, as shown in Figures 3c and 3d. The time to failure, t_f, is the elapsed time between the start of shear and the maximum shear stress. The maximum shear stress shall be plotted against the normal stress, as shown in Figure 4, and the strength envelope drawn to determine the drained or effective angle of internal friction, ϕ. In normally consolidated soils (see Fig. 4a), the strength envelope is based on normal stresses greater than any past or existing overburden pressure.

A brief description of undisturbed specimens should be given on the report form under "Remarks." The description should include color, approximate consistency, and any unusual features (such as stratification,

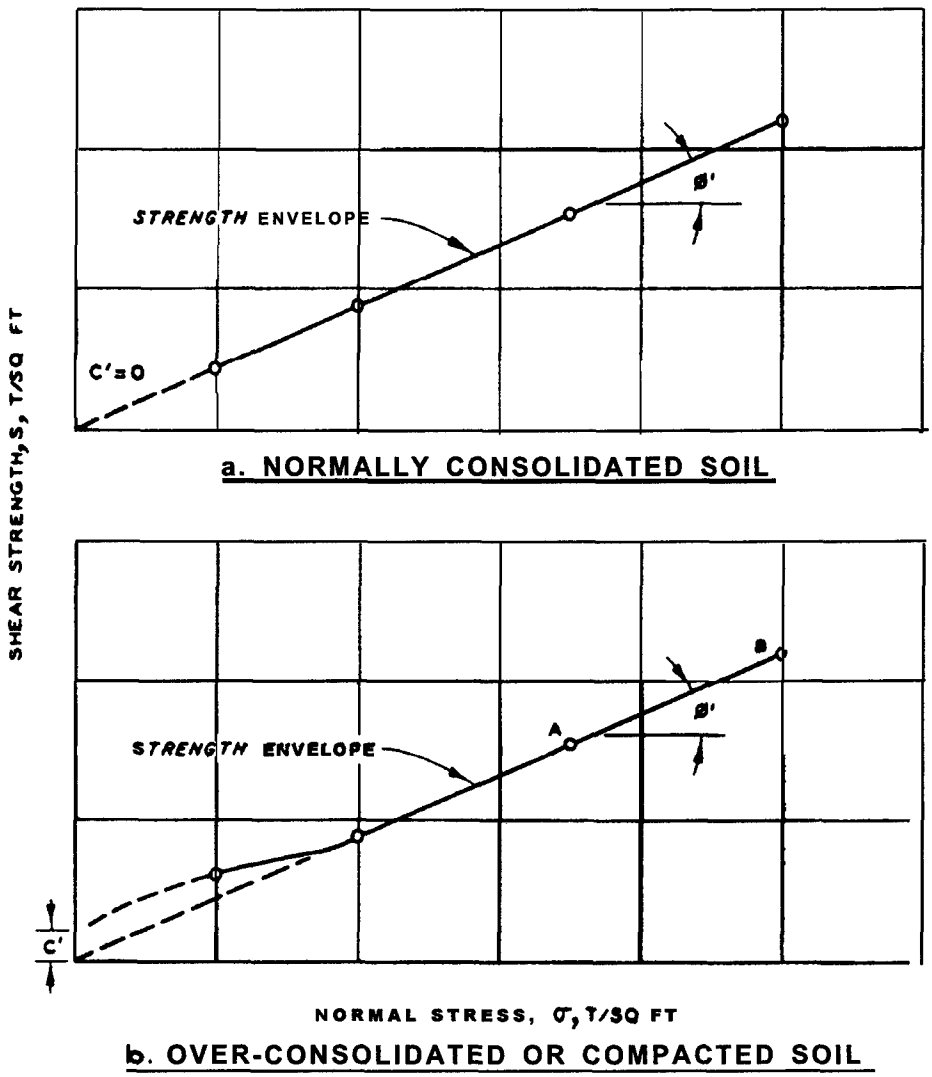


Figure 4. Examples of strength envelope s

fissures, roots, shells, sand pockets, etc.). For compacted specimens, give method of compaction used and the relation to maximum density and optimum water content.

7. SIMPLIFIED PROCEDURE FOR DRAINED (S) DIRECT SHEAR TESTS. In problems involving the long-term stability of fine-grained soils, the cohesion intercept, c', due to over-consolidation generally is ignored. The drained direct shear test may be performed by shearing duplicate specimens under the maximum normal stress expected in the

field. The strength envelope for determining the effective angle of internal friction, ϕ', shall be drawn as a straight line from the origin to the average value of the maximum shear stresses under this normal stress. Regardless of the magnitude of the expected normal stress, the normal stress used in this simplified procedure for testing compacted specimens must be at least 3 tons per sq ft to avoid the effects of prestressing caused by the compaction.

Although the testing of duplicate specimens under the same normal stress is satisfactory for relatively pervious or normally consolidated soils, it may not be conservative for preconsolidated clays. For the latter soils, the duplicate specimens should be sheared under different normal stresses, both of which are known to exceed the preconsolidation pressure, as illustrated by points A and B in Figure 4b.

d. POSSIBLE ERRORS: Following are possible errors that would cause inaccurate determinations of strength and stress-deformation characteristics:

a. Specimen disturbed while trimming. The trimming of specimens must be done in the humid room with every care taken to minimize disturbance of the natural soil structure or change in the natural water content. As a rule, the effect of trimming disturbance is inversely proportional to the size of the specimen.

b. Specimen disturbed while fitting into shear box. The specimen must exactly fit the inside of the shear box to insure complete lateral confinement, yet a pretrimmed specimen must be inserted without flexing or compressing. The specimen cutter must have the identical inside dimensions as those of the shear box.

c. Galvanic action in shear box. To prevent any change in the strength or stress-deformation characteristics due to galvanic currents in tests of long duration, all metal parts of the shear box should be of the same noncorrosive material.

d. Permeability of porous stones too low. Unless the porous

stones are frequently cleaned, they may become clogged by soil particles and full drainage of the specimen inhibited.

e. Slippage between porous stone and specimen. When testing undisturbed firm or stiff clays, particularly under low normal loads, it may not be possible to transfer the required shear force to the specimen by means of the standard porous stone. In such a case, slippage of the porous stone will result and a portion of the shear force will be applied to the specimen by the rear edge of the upper frame. The slippage may be marked by tilting of the upper frame and the development of an inclined shear plane through the upper rear corner rather than one through the midheight of the specimen. The use of dentated porous stones or of wire cloth or abrasive grit between the stone and the specimen may be necessary to effect the transfer of shear stress.

f. Rate of strain too fast. The time to failure in the drained (S) direct shear test must be long enough to achieve essentially complete dissipation of excess pore pressure at failure. The criterion given in paragraph 4c(3) should be considered as no more than an approximate guide to the minimum time to failure; twice this time may be necessary for some soils. In general, it is safer to shear too slowly.

DIRECT SHEAR TEST
(Specimen Data)

Date _____

Project _____

Boring No. _____ Sample No. _____

Shear Box No.				Normal Stress			T/sq ft.
Specimen No.				Classification			
		Before Test				After Test	
		Specimen		Trimmings		Specimen	
Tare No.		Cutter and Glass Plates					
Weight in grams: Tare plus wet soil							
Tare plus dry soil							
Water	W_w			W_{wo}		W_{wf}	
Tare							
Wet soil	W						
Dry soil	W_s						
Water content	w		%	w_o	%	w_f	%

Initial Condition of Specimen

Area in sq cm	A		Volume of solids in cc	V_s	
Height in cm	H_o		Void ratio = $(V_o - V_s) \div V_s$	e_o	
Volume in cc = A × H_o	V_o		Saturation, %	S_o	
Specific gravity of solids	G_s		Dry density in lb/cu ft	γ_d	

Condition of Specimen After Consolidation

Change in height during consolidation, in.	ΔH_o		Volume in cc = A × H_c	V_c	
Height in cm = H_o - 2.54ΔH_o	H_c		Void ratio = $(V_c - V_s) \div V_s$	e_c	

Condition of Specimen After Test

Change in height during shear test, in.	ΔH		Volume in cc = A × H_f	V_f	
Height in cm = H_c - 2.54ΔH	H_f		Void ratio = $(V_f - V_s) \div V_s$	e_f	
Saturation, %	S_f				

$$W_s = \frac{W}{1 + \frac{w_o}{100}} \quad , \quad V_s = \frac{W_s}{G_s} \quad , \quad S_o = \frac{\frac{w_o}{100} \times \frac{W_s}{\gamma_w}}{V_o - V_s} \times 100 \quad , \quad S_f = \frac{\frac{w_f}{100} \times \frac{W_s}{\gamma_w}}{V_f - V_s} \times 100 \quad , \quad \gamma_d = \frac{W_s}{V_o} \times 62.4$$

Remarks _____

_____

Technician _____ Computed by _____ Checked by _____

PLATE IX-1 ENG FORM 1 JUN 65 3850

DIRECT SHEAR TEST

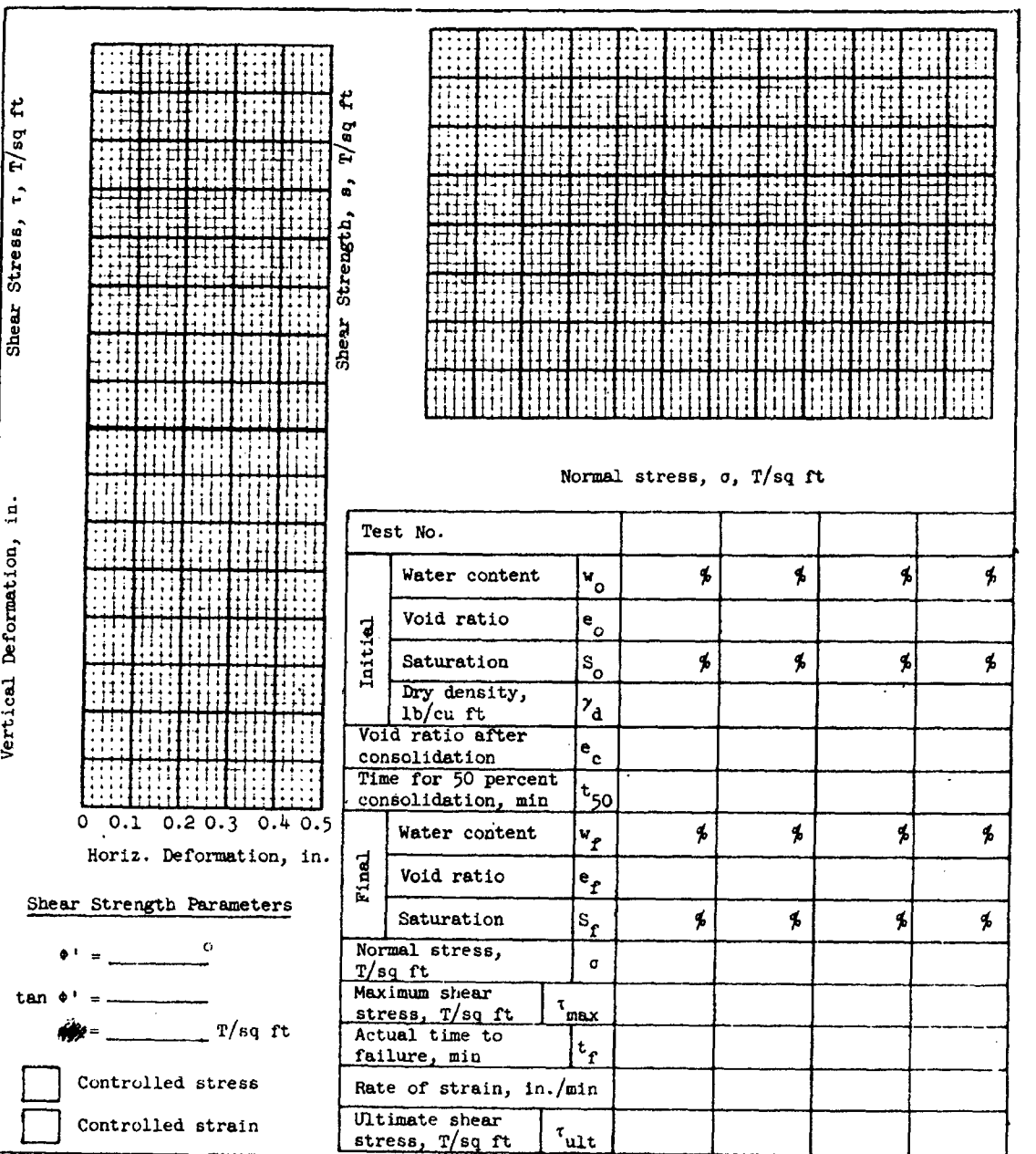
(Time-Shear Stress Data)

Date _____

Project _____

Boring No. _____ Sample No. _____ Normal Stress _____

Time	Elapsed Time min	Time Intervals min	Proving Ring Dial Reading 10^{-} in.	Shear Stress T/sq ft	Vertical Dial Reading 10^{-} in.	Horizontal Dial Reading 10^{-} in.



Shear Strength Parameters

ϕ' = _____ °

tan ϕ' = _____

= _____ T/sq ft

☐ Controlled stress

☐ Controlled strain

Test No.						
Initial	Water content	w_o	%	%	%	%
	Void ratio	e_o				
	Saturation	S_o	%	%	%	%
	Dry density, lb/cu ft	γ_d				
Void ratio after consolidation		e_c				
Time for 50 percent consolidation, min		t_{50}				
Final	Water content	w_f	%	%	%	%
	Void ratio	e_f				
	Saturation	S_f	%	%	%	%
Normal stress, T/sq ft		σ				
Maximum shear stress, T/sq ft		τ_{max}				
Actual time to failure, min		t_f				
Rate of strain, in./min						
Ultimate shear stress, T/sq ft		τ_{ult}				

Type of specimen		in. square	in. thick
Classification			
LL	PL	PI	G_s

Remarks _____

Project	
Area	
Boring No.	Sample No.
Depth El	Date
DIRECT SHEAR TEST REPORT	

PLATE IX-3 ENG FORM 1 JUN 65 2092

APPENDIX IXA:

DRAINED (S) REPEATED DIRECT SHEAR TEST

1. INTRODUCTION. The repeated direct shear test is used to measure the residual shear strength of a soil (primarily clay shale material) under drained conditions. In this test, a square soil specimen acted on by a normal stress is repeatedly sheared by reversal of the direction of shear until a minimum, i.e. residual, shear stress is determined. The concept of the test is that a specimen of material is acted upon by normal and shear stresses until, after large shear deformation, disaggregation and increase in parallel orientation of soil particles in the direction of shearing occur and a surface or a thin zone of remolded material is formed. Thus a minimum drained shearing resistance is offered. The shear surface or zone can be formed by (a) shearing an intact specimen, (b) cutting a plane surface through an intact specimen with a fine wire, or (c) shearing a completely remolded specimen. Practical laboratory considerations favor the use of a precut shear plane as it may not be possible to induce a plane surface by shearing an intact stiff-to-hard specimen. Any irregularities of the shear surface would introduce an added resistance that would not be a measure of the shear strength of the material.

2. APPARATUS. The apparatus may be similar to that described in Appendix IX, DRAINED DIRECT SHEAR TEST. A controlled strain device in which the direction of shear is capable of being reversed should be used to apply the horizontal shear force to the specimen.

3. SPECIMEN PREPARATION. a. Specimens should be prepared using procedures similar to those described in Appendix IX, DRAINED DIRECT SHEAR TEST; special saws may be required for trimming stiff-to-hard materials. Special care must be exercised to ensure that the specimen is not subjected to air-drying during or after trimming ope rations.

b. The specimen should consist of two pieces of intact material trimmed to fill the inside of the shear box or confining ring. The two pieces should be of approximately equal height and have a total height not in excess of 1 in. (preferably, the total height should be 0.5 in., but this is often not practical for stiff, fissured materials). The top and bottom surfaces of each piece should be plane and parallel. A close fit of each piece to the inside of the shear box is necessary. Stiff-to-hard materials may be cut to shape with a bandsaw or, in the case of very hard materials, with a diamond wheel.

c. From the soil trimings obtain about 200 g of material for water content and specific gravity determinations (see Appendix I, WATER CONTENT - GENERAL, and Appendix IV, SPECIFIC GRAVITY).

4. PROCEDURE, a. Preliminary. The procedure for setting up the test specimen shall consist of the following:

(1) Record all identifying information for the specimen, such as project title, boring number, sample number, and other pertinent data, on the data sheet (Plate IX-1 is a suggested form); note any difficulties encountered in preparation of the specimen. Determine the dimensions of the specimen and record as the initial dimensions of the specimen on the data sheet. Weigh and record the weight of specimen (plus tare, if used).

(2) The lower half of the specimen should be firmly seated against a saturated porous plate in the lower half of the shear box. A 0.010- to 0.020-in. projection of the lower half of the specimen above the lip of the box is desirable; certainly the top of this half of the specimen should not be initially below the lip. Then the upper half of the specimen should be placed in the shear box, the upper porous plate (saturated) added, and the remainder of the shear apparatus assembled.

A specimen of softer material may be precut inside the shear box. In this case, an intact specimen is firmly seated between saturated porous plates in the apparatus ; then, a plane should be cut with a small-diameter (0.008- to 0.014-in. diameter) steel wire through the

specimen at the separation between the upper and lower halves of the box. After cutting, the two halves of the specimen should be separated, and the cut surfaces inspected for planeness. Any irregularities should be removed with a straightedge.

(3) Place the shear box in position on the loading apparatus. At this time the upper and lower frames are in contact. Assemble the loading equipment and mount the two dial indicators or other deformation measuring apparatus to be used for measuring vertical and horizontal deformation. Both indicators should be set so that they can measure deformation in either direction.

b. Consolidation. The procedure for consolidating the specimen shall consist of the following:

(1) Apply the normal stress to the specimen without impact. A single value of normal stress may be used. A standard value of about 6 tons per sq ft is recommended as being high enough to prevent the swelling of most clay shale material yet low enough to minimize the problem of soil extruding from between the two halves of the box during shear. In addition, tests under higher normal stresses may be used to determine whether the strength envelope is a straight line.

(2) As soon as possible after applying the normal stress, fill the water reservoir with distilled or demineralized water to a point above the top of the specimen. Maintain this water level during the consolidation (or swell) and subsequent shear phases so that the specimen is at all times effectively submerged.

(3) The specimen should be allowed to consolidate or swell to an essentially equilibrium condition under the normal stress; a minimum period of 16 hr should be allowed before shear.

c. Repeated Shear Test. The procedure for shearing the specimen after consolidation shall consist of the following:

(1) A gap should be formed between the two halves of the box to ensure that normal and shear stresses are borne only by soil. This

gap should be kept between 0.015 to 0.025 in. to minimize extrusion of remolded soil from the shearing surface. Periodically during the test, the gap should be checked by inserting thickness gages, and adjusted as needed.

(2) Remove the alignment pins.

(3) Shearing displacement should be initiated at a controlled rate not in excess of 0.5 in. per day (about 0.00035 in. per min). Shear movement under constant normal stress should be continued with a reversal of direction after about 0.25-in. displacement to each side of the starting position until a minimum shearing resistance is attained. A semilogarithmic plot of shear stress (arithmetic scale) versus cumulative shear displacement (logarithmic scale) should be maintained during the test to show when a minimum value has been reached; only the shear stress measured at the midpoint of each shearing stroke (that is, when the two halves of the shear box are aligned vertically) should be plotted.

(4) Observations of vertical and horizontal deformations and the applied shear force sufficient to define the stress deformation curve for each shearing stroke shall be made.

(5) If, after the standard test is complete, the effects of increased normal stress and decreased displacement rate are to be studied, this information should also be obtained according to uniform procedures.

First, the normal stress should be doubled while the two halves of the shear box are vertically aligned, and the specimen should be permitted to come to equilibrium (minimum of 16 hr). Shear displacement should be initiated at a rate of about 0.5 in. per day and continued until a minimum shearing resistance is reached.

Second, the effect of decreased rate of displacement should be determined as follows. After the minimum shearing resistance is reached under the high normal stress (approximately 12 tons per sq ft) and at a displacement rate of about 0.5 in. per day, the rate of displacement

should be reduced to a tenth of the standard rate (that is, to about 0.05 in. per day) without any change in the normal stress. The rate of displacement should be reduced soon after the upper half of the shear box has passed through the initial, vertically aligned position. Shearing with repeated reversals of direction as described above should be continued until a minimum shearing resistance is reached.

(6) Remove the specimen from the shear box, blot any excess moisture, and determine the water content of the specimen† (see Appendix I, WATER CONTENT - GENERAL). The dry weight of the specimen should be computed using the water content based on specimen trimmings taken during specimen preparation.

5. COMPUTATIONS. See Appendix IX, DRAINED (S) DIRECT SHEAR TEST.

6. PRESENTATION OF RESULTS. Report forms have not been standardized; the method of presentation of results will be determined by the project (design) engineer requesting the tests.

7. POSSIBLE ERRORS. The following are possible errors that may cause inaccurate determinations of strength and stress-deformation characteristics:

a. Air -Drying of Specimen During Preparation. The trimming of specimens should be done in a humid room with every precaution taken to prevent change in natural water content. Air-drying may cause the specimen to slake readily when inundated with water, and thus change the strength or stress-deformation characteristics.

b. Top and Bottom Surfaces of Each Half of a Specimen Not Plane and Parallel. Irregular surfaces may introduce a geometric component to the measured shearing resistance.

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† If considerable remolded material exists in the shear zone, it should be removed and its water content should be determined in addition to that of the remainder of the specimen.

c. Too Large a Gap. Maintaining too large a gap between the upper and lower frames of the shear box may result in excessive extrusion of the specimen.

d. Absence of Gap. A gap must be maintained throughout the test to prevent the normal load from being borne by the lower frame of the shear box.

e. Inaccurate Measurement of Shear Stress. Because of the very small shear resistances offered by some clay shale materials, measurement of shear stress must be very precise.

f. Permeability of Porous Stones Too Low. Unless the porous stones are frequently cleaned, they may become clogged by soil particles and ingress or egress of water to or from the specimen may be inhibited.

g. Galvanic Action in Shear Box. To prevent any change in the strength or stress-deformation characteristics due to galvanic currents in tests of long duration, all metal parts of the shear box should be constructed of the same noncorrosive material.

h. Stopping Test Too Soon. The test must be carried to a cumulative shear deformation sufficient to establish that the minimum shear resistance offered by a specimen under a given normal stress has indeed been determined. A semilogarithmic plot of shear stress (arithmetic scale) versus cumulative shear displacement (logarithmic scale) is essential in making this determination.

APPENDIX X:

TRIAXIAL COMPRESSION TESTS

1. PRINCIPLES OF THE TRIAXIAL COMPRESSION TEST. The triaxial compression test is used to measure the shear strength of a soil under controlled drainage conditions. In the basic triaxial test, a cylindrical specimen of soil encased in a rubber membrane is placed in a triaxial compression chamber, subjected to a confining fluid pressure, and then loaded axially to failure. Connections at the ends of the specimen permit controlled drainage of pore water from the specimen. The procedures presented herein apply only to the basic test conducted with limited drainage conditions, and do not include special types or variants of this test. In general, a minimum of three specimens, each under a different confining pressure, are tested to establish the relation between shear strength and normal stress. The test is called "triaxial" because the three principal stresses are known and controlled. Prior to shear, the three principal stresses are equal to the chamber fluid pressure. During shear, the major principal stress, σ_1, is equal to the applied axial stress (P/A) plus the chamber pressure, σ_3 (see Fig. 1). The applied axial stress, $\sigma_1 - \sigma_3$, is termed the "deviator stress." The

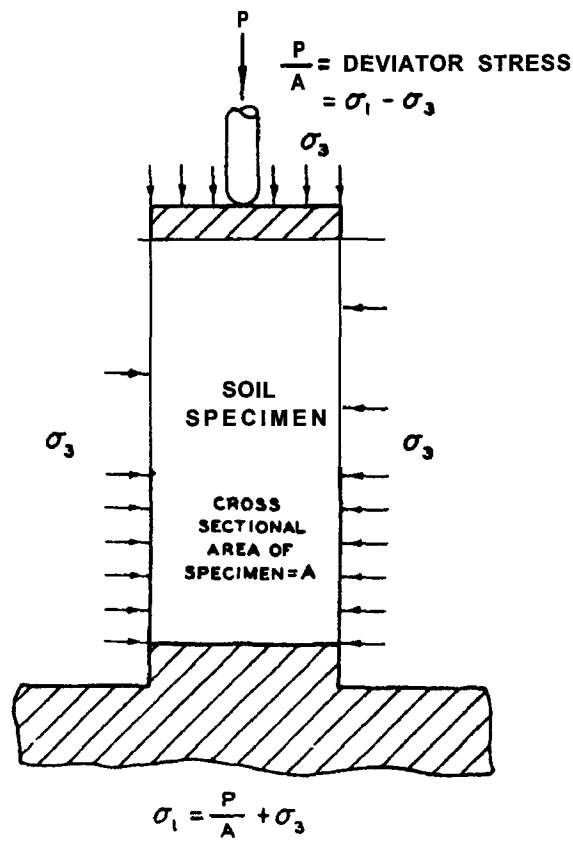


Figure 1. Diagram showing stresses during triaxial compression test

intermediate principal stress, σ_2, and the minor principal stress, σ_3, are identical in the test, and are equal to the confining or chamber pressure hereafter referred to as σ_3.

A soil mass may be considered as a compressible skeleton of solid particles. In saturated soils the void spaces are completely filled with water; in partially saturated soils the void spaces are filled with both water and air. Shear stresses are carried only by the skeleton of solid particles, whereas the normal stress on any plane is carried by both the solid particles and the pore water. In a triaxial test, the shear strength is determined in terms of the total stress (intergranular stress plus pore water pressure), unless (a) complete drainage is provided during the test so that the pore pressure is equal to zero at failure, or (b) measurements of pore pressure are made during the test. When the pore pressure at failure is known, the shear strength can be computed in terms of the stress carried by the soil particles (termed effective or intergranular stress). In recent years, significant advances have been made in the techniques of measuring pore pressures in the triaxial test and in the interpretation of the data obtained; however, difficulties still exist in this respect. Pore pressure measurements during shear are seldom required in routine investigations, as the basic triaxial tests are sufficient to furnish shear strengths for the limiting conditions of drainage. Procedures for measuring pore pressures in the triaxial test during shear are discussed elsewhere† and are beyond the scope of this manual.

2. TYPES OF TESTS. The three types of basic triaxial compression tests are unconsolidated-undrained, consolidated-undrained, and consolidated-drained, subsequently referred to as the Q, R, and S tests, respectively. As these names imply, they are derived from the drainage conditions allowed to prevail during the test. The type of test is selected

† A. W. Bishop and D. J. Henkel, The Measurement of Soil Properties in the Triaxial Test, 2nd ed. (London, Edward Arnold Ltd., 1962).

to closely simulate, or to bracket, the conditions anticipated in the field. In the basic tests, the initial principal stresses are equal; that is, no attempt is made to duplicate stress systems in the field in which the principal stresses are not equal.

a. Q Test. In the Q test the water content of the test specimen is not permitted to change during the application of the confining pressure or during the loading of the specimen to failure by increasing the deviator stress. The Q test is usually applicable only to soils which are not free-draining, that is, to soils having a permeability less than 10×10^{-4} cm per sec.

b. R Test. In the R test, complete consolidation of the test specimen is permitted under the confining pressure. Then, with the water content held constant, the specimen is loaded to failure by increasing the deviator stress. Specimens must as a general rule be completely saturated before application of the deviator stress.

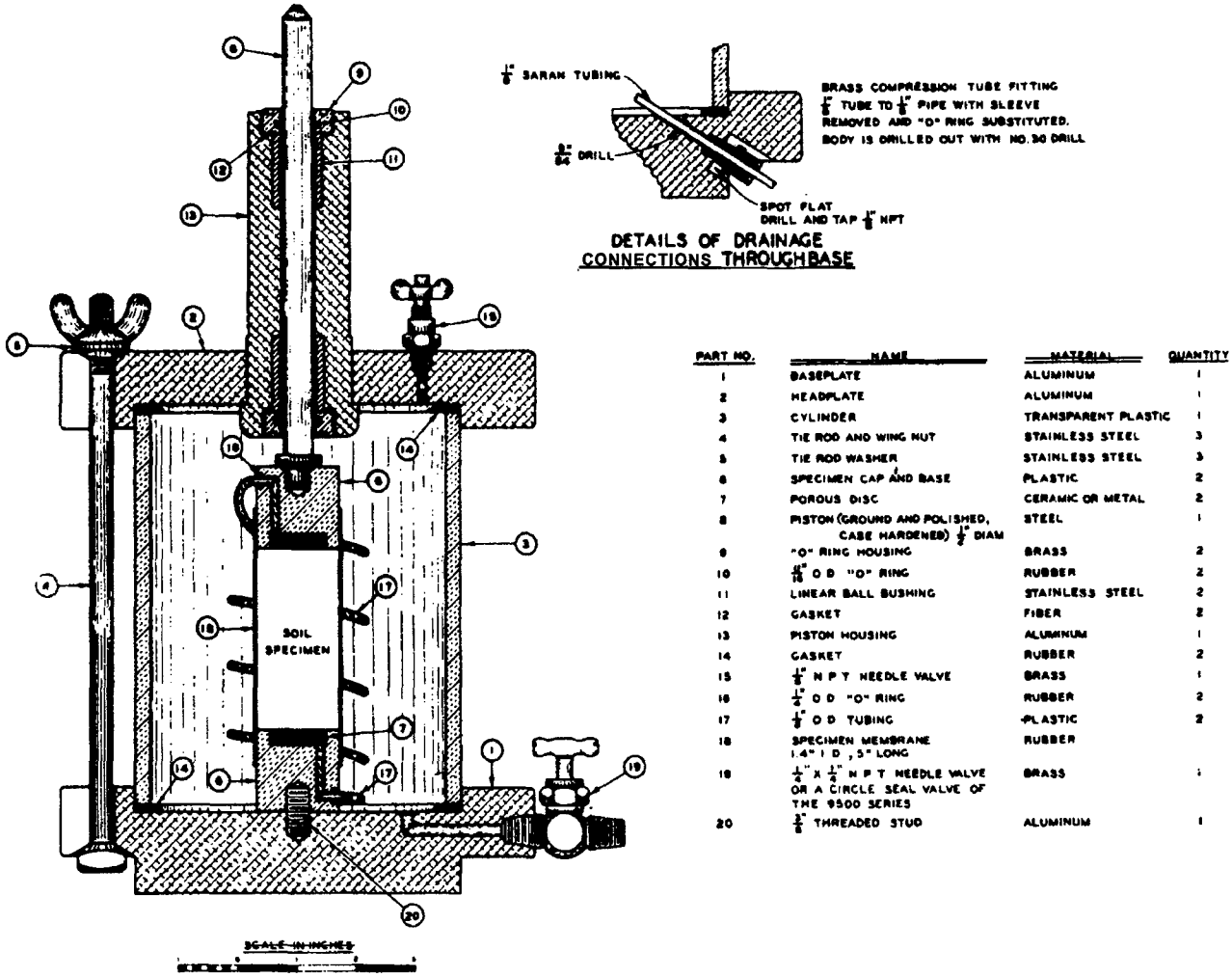
c. S Test. In the S test, complete consolidation of the test specimen is permitted under the confining pressure and during the loading of the specimen to failure by increasing the deviator stress. Consequently, no excess pore pressures exist at the time of failure.

3. APPARATUS. a. Loading Devices. Various devices may be used to apply axial load to the specimen. These devices can be classified as either apparatus in which axial loads are measured outside the triaxial chamber or apparatus in which axial loads are measured inside the triaxial chamber by using a proving ring or frame, an electrical transducer, or a pressure capsule. Any- equipment used should be calibrated to permit determination of loads actually applied to the soil specimen.

Loading devices can be further grouped under controlled-strain or controlled-stress types. In controlled-strain tests, the specimen is strained axially at a predetermined rate; in controlled-stress tests, predetermined increments of load are applied to the specimen at fixed intervals of time. Controlled-strain loading devices, such as commercial

testing machines, are preferred for short-duration tests using piston-type test apparatus. If available, an automatic stress-strain recorder may be used to measure and record applied axial loads and strains.

b. Triaxial Compression Chamber. The triaxial compression chamber consists primarily of a headplate and a baseplate separated by a transparent plastic cylinder.* A drawing of a typical triaxial compression chamber for 1.4-in. -diameter specimens is shown in Figure 2. Chamber



PART NO.	NAME	MATERIAL	QUANTITY
1	BASEPLATE	ALUMINUM	1
2	HEADPLATE	ALUMINUM	1
3	CYLINDER	TRANSPARENT PLASTIC	1
4	TIE ROD AND WING NUT	STAINLESS STEEL	3
5	TIE ROD WASHER	STAINLESS STEEL	3
6	SPECIMEN CAP AND BASE	PLASTIC	2
7	POROUS DISC	CERAMIC OR METAL	2
8	PISTON (GROUND AND POLISHED, CASE HARDENED) 1/2" DIAM	STEEL	1
9	"O" RING HOUSING	BRASS	2
10	1/16" O D "O" RING	RUBBER	2
11	LINEAR BALL BUSHING	STAINLESS STEEL	2
12	GASKET	FIBER	2
13	PISTON HOUSING	ALUMINUM	1
14	GASKET	RUBBER	2
15	1/8" N P T NEEDLE VALVE	BRASS	1
16	1/4" O D "O" RING	RUBBER	2
17	1/8" O D TUBING	PLASTIC	2
18	SPECIMEN MEMBRANE 1.4" I D , 5" LONG	RUBBER	
19	1/4" X 1/4" N P T NEEDLE VALVE OR A CIRCLE SEAL VALVE OF THE 9500 SERIES	BRASS	1
20	1/8" THREADED STUD	ALUMINUM	1

Figure 2. Details of typical triaxial compression chamber

* Adequate safety precautions should be taken, or the transparent plastic cylinder should be replaced by a metal cylinder, if chamber pressures in excess of 100 psi are used.

dimensions and type will vary depending on the size of specimen tested and on pressure and load requirements. The baseplate has one inlet through which the pressure liquid is supplied to the chamber and two inlets leading to the specimen base and cap to permit saturation and drainage of the specimen when required, The headplate has a vent valve so that air can be forced out of the chamber as it is filled with the pressure fluid. The cylinder is held tightly against rubber gaskets by bolts or tie rods connecting the headplate and baseplate.

In piston-type test apparatus in which the axial, loads are measured outside the triaxial compression chamber, piston friction can have a significant effect on the indicated applied load, and measures should be taken to reduce friction to tolerable limits. Pistons generally should consist of ground and polished case-hardened steel rods with diameters between 1/4 and 1/2 in. for testing 1.4-in.-diameter specimens; heavier pistons are required for larger specimens. The following measures have been found to reduce piston friction to tolerable amounts.

(1) The use of linear ball bushings as shown in Figure 2. The unique design of these bushings permits unlimited axial movement of the piston with a minimum of friction. Leakage around the piston is reduced by means of Q-rings, Quad-rings, flexible diaphragms, or other devices. A seal incorporating Q-rings is shown in Figure 2. The benefi - cial effects of using linear ball bushings in comparison with steel bushings are demonstrated by the data shown in Figure 3. The amount of lateral force

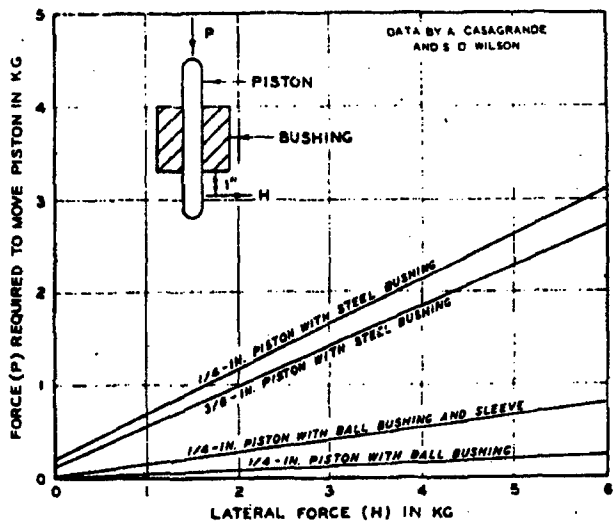


Figure 3. Effect of lateral force on piston friction in triaxial compression apparatus

transmitted to the piston, if the specimen cap tends to tilt during a test, cannot be determined; however, the data shown in Figure 3 indicate that the resulting piston friction would be negligible even for relatively large lateral forces.

(2) Rotation of the piston within the bushings during the application of the deviator stress. (Commercial devices are available to rotate the piston during the test.) This method is very effective in reducing friction; however, a more complex design of the specimen cap is necessary, and unless the piston is rotated continuously, appreciable friction would still develop during longtime tests. When linear ball bushings are used, the piston should never be rotated except under special conditions designated by the manufacturer.

Although these measures will reduce piston friction to negligible amounts during the course of the test, it is always preferable to measure the actual piston friction before the start of the test. This can readily be done by starting the axial load application with the bottom of the piston raised slightly above the top of the specimen cap. Thus any starting friction or residual friction, as indicated by the load necessary to move the piston down into contact with the cap, can be subtracted from the measured load.

c. Specimen Caps and Bases. Specimen caps and bases should be constructed of a lightweight noncorrosive material and should be of the same diameter as the test specimen in order to avoid entrapment of air at the contact faces. Solid caps and bases should be used for the Q test to prevent drainage of the specimens. Caps and bases with porous metal or porous stone inserts and drainage connections, as shown in Figure 4, should be used for the R and S tests. The porous inserts should be more pervious than the soil being tested to permit effective drainage. For routine testing, stones of medium porosity are satisfactory. The specimen cap should be designed to permit slight tilting with the piston in contact position, as shown in Figure 4.

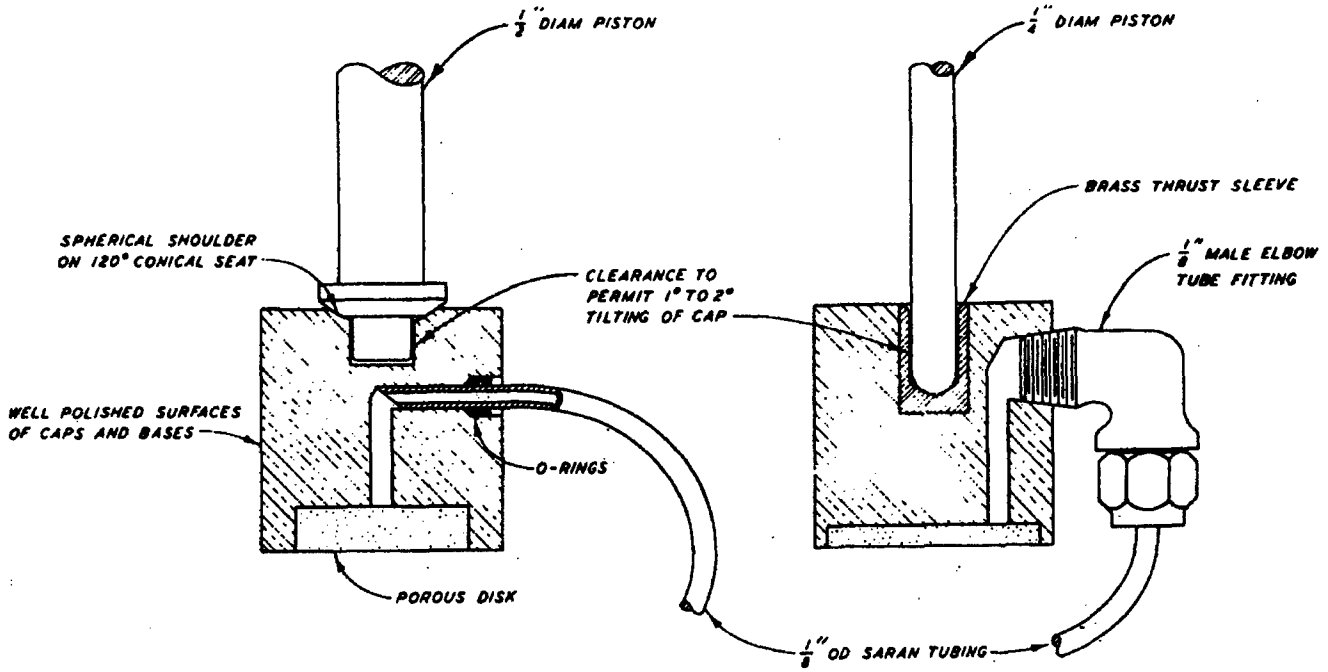


Figure 4. Details of typical 1.4-in.-diameter specimen caps showing drainage connections and piston seats

d. Rubber Membranes. Rubber membranes used to encase the specimen should provide reliable protection against leakage, yet offer minimum restraint to the specimen. Commercially available rubber membranes having thicknesses ranging from 0.0025 in. (for soft clays) to 0.010 in. (for sands or for clays containing sharp particles) are generally satisfactory for 1.4-in.-diameter specimens. Rubber membranes about 0.010 in. or greater in thickness are suitable for larger specimens. Membranes should be carefully inspected prior to use, and if any flaws or pinholes are evident, the membranes should be discarded. The use of two thin membranes separated by a thin film of silicone grease will afford protection against leakage through an undetected pinhole and will minimize the possibility of air leakage from the chamber fluid into the specimen during tests of relatively long duration. Since no rubber membrane is

completely impervious, the use of special membranes or chamber fluids† may sometimes be necessary, such as during periods of undrained shear that exceed a few hours. The membrane is sealed against the cap and base by rubber O-rings or rubber bands. Leakage around the ends of the membrane, where it is sealed against the cap and the base, as well as through fittings, valves, etc., can develop unless close attention is given to details in the manufacture and use of the apparatus.†

e. Equipment for Preparing Specimens. (1) Cohesive soils. A specimen trimming frame is recommended for preparing specimens of most cohesive soils. The specimen is held in a vertical position between two circular plates containing pins which press into the ends of the specimen to prevent movement during trimming. The edges of the trimming frame act as vertical guides for the cutting equipment and control the final diameter of the specimen. Details of a typical trimming frame for 1.4-in.-diameter specimens are shown in Figure 5. Wire saws and knives of various sizes and types are used with the trimmer (see Fig. 7, p. 12). Split or solid cylinders with a beveled cutting edge can also be used to trim specimens. The use of a motorized soil lathe may be advantageous in reducing the time required for preparing specimens of certain types of soils. A miter box or cradle (see Fig. 8, p. 13) is required to trim the specimen to a fixed length and to insure that the ends of the specimen are parallel with each other and perpendicular to the axis of the specimen.

(2) Cohesionless soils. A forming jacket consisting of a split mold which incloses a rubber membrane is required for cohesionless soils. The inside diameter of the mold minus the double thickness of the membrane is equal to the diameter of the specimen required. A funnel or special spoon (see Fig. 5 of Appendix VII, PERMEABILITY TESTS) for placing the material inside the jacket and a tamping hammer or vibratory

† S. J. Poulos, Report on Control of Leakage in the Triaxial Test, Soil Mechanics Series No. 71, Harvard University (Cambridge, Mass., March 1964).

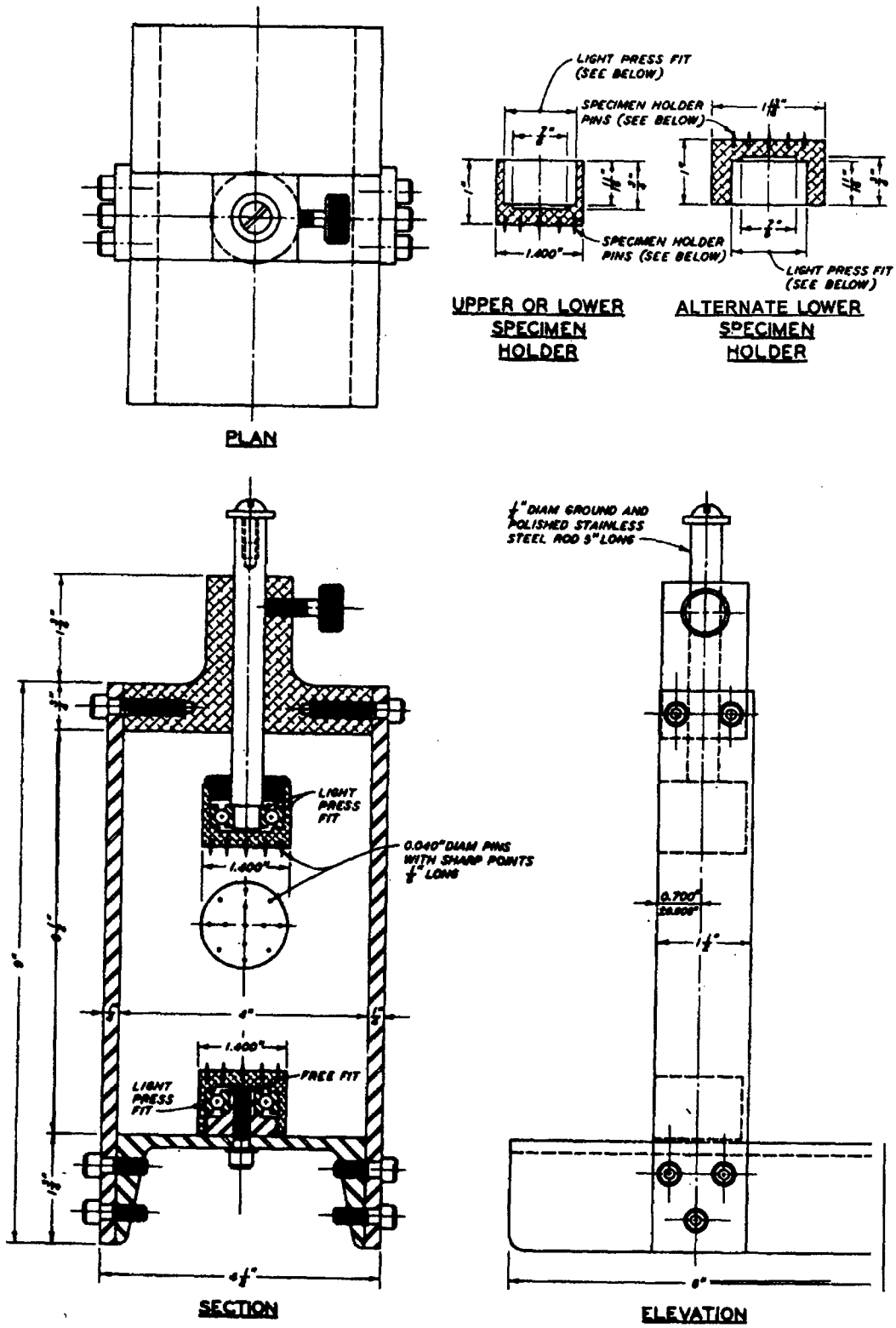


Figure 5. Details of a trimming frame for preparing 1.4-in.-diameter specimens

equipment for compacting the material are necessary.

(3) Soils containing gravel. Large-size forming jackets, the dimensions of which will depend on specimen size requirements subsequently described, are necessary for preparing specimens of material containing gravel. Special compacting equipment is also necessary for such soils, depending on the type of soil and the procedures used.

f. Equipment for Using Back Pressure to Saturate Specimens. Special equipment required for saturating specimens by using back pressures is described in paragraph 6a.

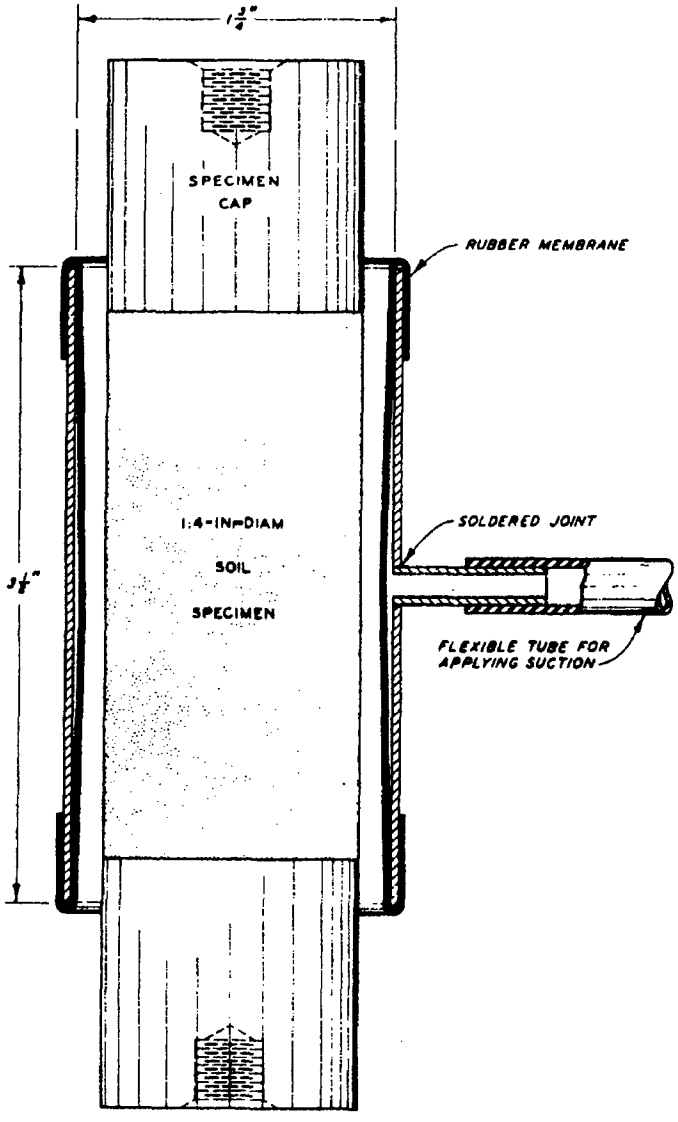


Figure 6. Details of a membrane stretcher for 1.4-in.-diameter specimens

g. Miscellaneous Equipment. Other items of equipment needed for the triaxial compression tests are as follows:

(1) Membrane stretcher. A cylindrical tube, larger in diameter than the soil specimen, which has a tube connected to its side for application of a vacuum. Details of a membrane stretcher for 1.4-in.-diameter specimens are shown in Figure 6.

(2) Pressure reservoir, generally a metal tank. The reservoir is filled with the fluid (usually deaired water) for applying the chamber pressure and is provided with a pressure regulator and a Bourdon gage. The

regulator should be capable of controlling pressures to within ±1/2 percent, though more precise methods of controlling and maintaining chamber pres - sures are required for tests of long duration.

(3) Measuring equipment, such as dial indicators and calipers. Precise instruments should be used for measuring the dimensions of a specimen with the desired accuracy.

(4) Deaired water, distilled or demineralized.

(5) Vacuum and air pressure supply.

(6) Bourdon gages of various sizes and capacities.

(7) A timing device, either a watch or clock with second hand.

(8) Balances, sensitive to 0.01 g and to 0.1 g.

(9) Apparatus, necessary to determine water content and specific gravity (see Appendices I, WATER CONTENT - GENERAL, and IV, SPECIFIC GRAVITY).

4. PREPARATION OF SPECIMENS. Specimens shall have an initial height of not less than 2. 1 times the initial diameter, though the minimum initial height of a specimen must be 2.25 times the diameter if the soil contains particles retained on the No. 4 sieve. The maximum particle size permitted in any specimen shall be no greater than one-sixth of the specimen diameter. Triaxial specimens i.4, 2.8, 4, 6, 12, and 15 in. in diameter are most commonly used.

a. Cohesive Soils Containing Negligible Amounts of 'Gravel. Specimens i.4 in. in diameter are generally satisfactory for testing cohesive soils containing a negligible amount of gravel, while specimens of larger diameter may be advisable for undisturbed soils having marked stratification, fissures, or other discontinuities. Depending on the type of sample, specimens shall be prepared by either of the following procedures:

(1) Trimming specimens of cohesive soil. A sample that is uniform in character and sufficient in amount to provide a minimum of three specimens is required. For undisturbed soils, samples about 5 in. in diameter are preferred for triaxial tests using 1.4-in.-diameter specimens.

Specimens shall be prepared in a humid room and tested as soon as possible thereafter to prevent evaporation of moisture. Extreme care shall be taken in preparing the specimens to preclude the least possible disturbance to the structure of the soil. The specimens shall be prepared as follows:

(a) Cut a section of suitable length from the sample. As a rule, the specimens should be cut with the long axes parallel to the long axis of the sample; any influence of stratification is commonly disregarded. However, comparative tests can be made, if necessary, to determine the effects of stratification. When a 5-in.-diameter undisturbed sample is to be used for 1.4-in.-diameter specimens, cut the sample axially into quadrants using a wire saw or other convenient cutting tool. Use three of the quadrants for specimens; seal the fourth quadrant in wax and preserve it for a possible check test.

(b) Carefully trim each specimen to the required diameter, using a trimming frame or similar equipment (see Fig. 7). Use one side of the trimming frame for

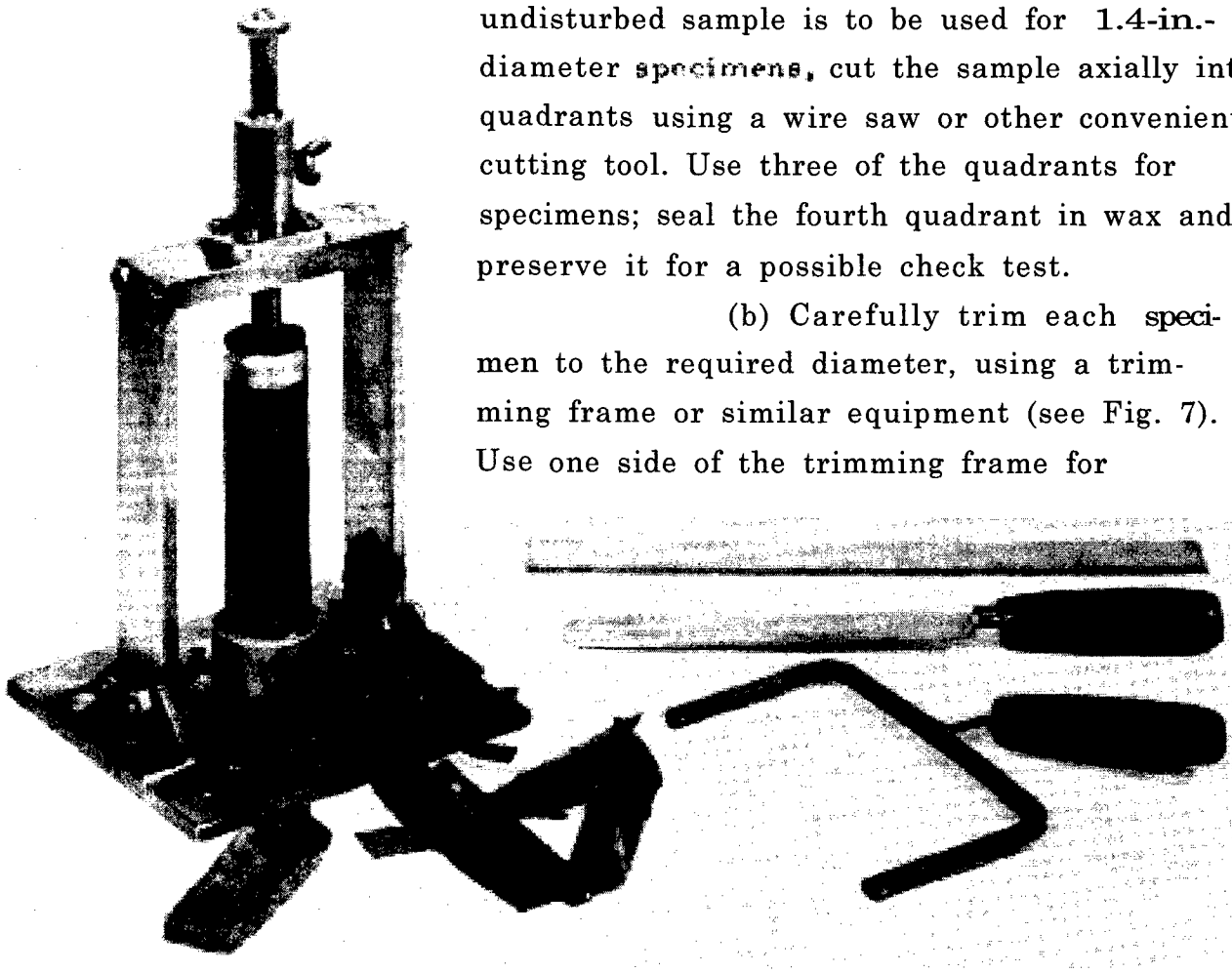


Figure 7. Prepared triaxial specimen, trimming frame, and cutting tools

preliminary cutting, and the other side for final trimming. A specimen after trimming is also shown in Figure 7. Ordinarily, the specimen is trimmed by pressing the wire saw or trimming knife against the edges of the frame and cutting from top to bottom. In trimming stiff or varved clays, move the wire saw from the top and bottom toward the middle of the specimen to prevent breaking off pieces at the ends. Remove any small shells or pebbles encountered during the trimming operations. Carefully fill voids on the surface of the specimen with remolded soil obtained from the trimmings. Cut specimen to the required length (usually 3 to 3-1/2 in. for 1.4-in. -diameter specimens and 6 to 7 in. for 2.8-in. -diameter specimens) using a miter box, as shown in Figure 8

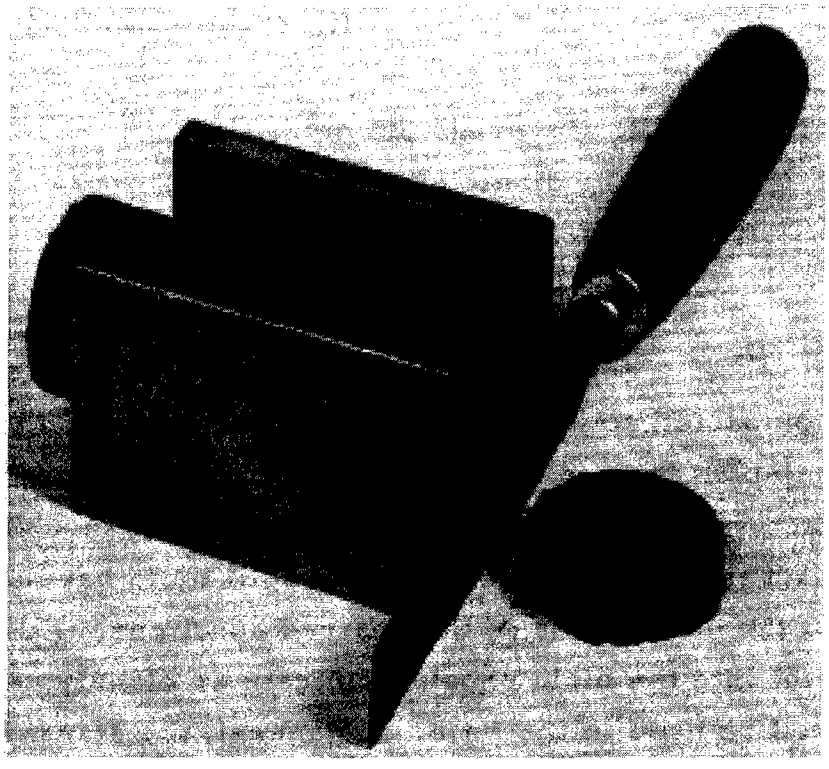


Figure 8. Squaring ends of specimen with a miter box

(c) From the soil trimmings, obtain 200 g of material for specific gravity and water content determinations (see Appendixes I, WATER CONTENT - GENERAL, and IV, SPECIFIC GRAVITY).

(d) Weigh the specimen to an accuracy of ±0.01 g for 1.4-in. -diameter specimens and ±0.1 g for 2.8-in.-diameter specimens.

(e) Measure the height and diameter of the specimen to an accuracy of ±0.01 in. Specimen dimensions based on measurements of the

trimming frame guides and miter box length are not sufficiently accurate. The average height, H_o, of the specimen should be determined from at least four measurements, while the average diameter should be determined from measurements at the top, center, and bottom of the specimen, as follows:

$$D_o = \frac{D_t + 2D_c + D_b}{4}$$

where D_o = average diameter
D_t = diameter at top
D_c = diameter at center
D_b = diameter at bottom

(2) Compacting specimens of cohesive soil. Specimens of compacted soil may be trimmed, as described in paragraph 4a(1), from samples formed in a compaction mold (a 4-in.-diameter sample is satisfactory for 1.4-in.-diameter specimens), though it is preferable to compact individual specimens in a split mold having inside dimensions equal to the dimensions of the desired specimen. The method of compacting the soil into the mold should duplicate as closely as possible the method that will be used in the field. In general, the standard impact type of compaction (see Appendix VI, COMPACTION TESTS) will not produce the same soil structure and stress-deformation characteristics as the kneading action of the field compaction equipment. Therefore, the soil should preferably be compacted into the mold (whether a specimen-size or a standard compaction mold) in at least six layers, using a pressing or kneading action of a tamper having an area in contact with the soil of less than one-sixth the area of the mold, and thoroughly scarifying the surface of each layer before placing the next. The sample shall be prepared according to paragraph 2b of Appendix VI, COMPACTION TESTS, thoroughly mixed with sufficient water to produce the desired water content, and then

stored in an airtight container for at least 16 hr. The desired density may be produced by either (1) kneading or tamping each layer until the accumulative weight of soil placed in the mold is compacted to a known volume or (2) adjusting the number of layers, the number of tamps per layer, and the force per tamp. For the latter method of control, special constant-force tampers (such as the Harvard miniature compactor for 1.4-in.-diameter specimens† or similar compactors for 2.8-in.-diameter and larger specimens‡) are necessary. After each specimen compacted to finished dimensions has been removed from the mold, proceed in accordance with steps (c) through (e) of paragraph 4a(1).

b. Cohesionless Soils Containing Negligible Amounts of Gravel. Soils which possess little or no cohesion are difficult if not impossible to trim into a specimen. If undisturbed samples of such materials are available in sampling tubes, satisfactory specimens can usually be obtained by freezing the sample to permit cutting out suitable specimens. Samples should be drained before freezing. The frozen specimens are placed in the triaxial chamber, allowed to thaw after application of the chamber pressure, and then tested as desired. Some slight disturbance probably occurs as a result of the freezing, but the natural stratification and structure of the material are retained. In most cases, however, it is permissible to test cohesionless soils in the remolded state by forming the specimen at the desired density or at a series of densities which will permit interpolation to the desired density. Specimens prepared in this

† A. Casagrande, J. M. Corso, and S. D. Wilson, Report to Waterways Experiment Station on the 1949-1950 Program of Investigation of Effect of Long-Time Loading on the Strength of Clays and Shales at Constant Water Content, Harvard University (Cambridge, Mass., July 1950).

‡ A. Casagrande and R. C. Hirschfeld, Second Progress Report on Investigation of Stress-Deformation and Strength Characteristics of Compacted Clays, Soil Mechanics Series No. 65, Harvard University (Cambridge, Mass., April 1962).

manner should generally be 2.8 in. in diameter or larger, depending on the maximum particle size. The procedure for forming the test specimen shall consist of the following steps:

(1) Oven-dry and weigh an amount of material sufficient to provide somewhat more than the desired volume of specimen.

(2) Place the forming jacket, with the membrane inside, over the specimen base of the triaxial compression device.

(3) Evacuate the air between the membrane and the inside face of the forming jacket.

(4) After mixing the dried material to avoid segregation, place the specimen, by means of a funnel or the special spoon, inside the forming jacket in equal layers. For 2.8-in.-diameter specimens, 10 layers of equal thickness are adequate. Starting with the bottom layer, compact each layer by blows with a tamping hammer, increasing the number of blows per layer linearly with the height of the layer above the bottom layer.† The total number of blows required for a specimen of a given material will depend on the density desired. Considerable experience is usually required to establish the proper procedure for compacting a material to a desired uniform density by this method. A specimen formed properly in the above-specified manner, when confined and axially loaded, will deform symmetrically with respect to its midheight, indicating that a uniform density has been obtained along the height of the specimen.

(5) As an alternate procedure, the entire specimen may be placed in a loose condition by means of a funnel or special spoon. The desired density may then be achieved by vibrating the specimen in the forming jacket to obtain a specimen of predetermined height and corresponding density. A specimen formed properly in this manner, when

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† Liang-Sheng Chen, "An investigation of stress strain and strength characteristics of cohesionless soils by triaxial compression tests," Proceedings, Second International Conference on Soil Mechanics and Foundation Engineering, vol. V (Rotterdam, 1948), pp. 35-43.

confined and axially loaded, will deform symmetrically with respect to its midheight.

(6) Subtract weight of unused material from original weight of the sample to obtain weight of material in the specimen.

(7) After the forming jacket is filled to the desired height, place the specimen cap on the top of the specimen, roll the ends of the, membrane over the specimen cap and base, and fasten the ends with rubber bands or O-rings. Apply a low vacuum to the specimen through the base and remove the forming jacket.

(8) Measure height and diameter as specified in paragraph 4a(1)(e).

c. Soils Containing Gravel. The size of specimens containing appreciable amounts of gravel is governed by the requirements of paragraph4. If the material to be tested is in an undisturbed state, the specimens shall be prepared according to the applicable requirements of paragraphs 4a and 4b, with the size of specimen based on anestimate of the largest particle size. In testing compacted soils, the largest particle size is usuallyknown, and the entire sample should be tested, whenever possible, without removing any of the coarser particles. However, it may be necessaryto remove the particles larger than a certain size to comply with the requirements for specimen size, though such practice will result in lower measured values of the shear strength and should be avoided if possible. Oversize particles should be removed and, if comprising more than 10 percent by weight of the sample, be replaced by anequalpercentage by weight of material retained on the No. 4 sieve and passing the maximum allowable sieve size. The percentage of material finer than the No. 4 sieve thus remains constant (see paragraph 2b of Appendix VI, COMPACTION TESTS). It will generally be necessary to prepare compacted samples of material containing gravel inside a forming jacket placed on the specimen base. If the material is cohesionless, it should be oven-dried and compacted in layers inside the membrane and forming jacket using the procedure in paragraph 4b as a guide. When specimens of very high density are required, the

samples should be compacted preferably by vibration to avoid rupturing the membrane. The use of two membranes will provide additional insurance against possible leakage during the test as a result of membrane rupture. If the sample contains a significant amount of fine-grained material, the soil usually must possess the proper water content before it can be compacted to the desired density. Then, a special split compaction mold is used for forming the specimen. The inside dimensions of the mold are equal to the dimensions of the triaxial specimen desired. No membrane is used inside the mold, as the membrane can be readily placed over the compacted specimen after it is removed from the split mold. The specimen should be compacted to the desired density in accordance with paragraph 4a(2).



Figure 9. Placing rubber membrane over a 2.8-in. -diameter specimen using a membrane stretcher

5. Q TEST. a. Procedure. The procedure for the Q test shall consist of the following steps:

(1) Record all identifying information for the sample project number or name, boring number, and other pertinent data, on a data sheet (see Plate X-1).

(2) Place one of the prepared specimens on the base.

(3) Place a rubber membrane (see Fig. 9) in the

membrane stretcher, turn both ends of the membrane over the ends of the stretcher, and apply a vacuum to the stretcher. Carefully lower the stretcher and membrane over the specimen as shown in Figure 9. Place the specimen cap on the top of the specimen and release the vacuum on the membrane stretcher. Turn the ends of the membrane down around the base and up around the specimen cap and fasten the ends with O-rings or rubber bands. With 1.4-in. -diameter specimens of relatively insensitive soils, it is easier to roll the membrane over the specimen as shown in Figure 10.

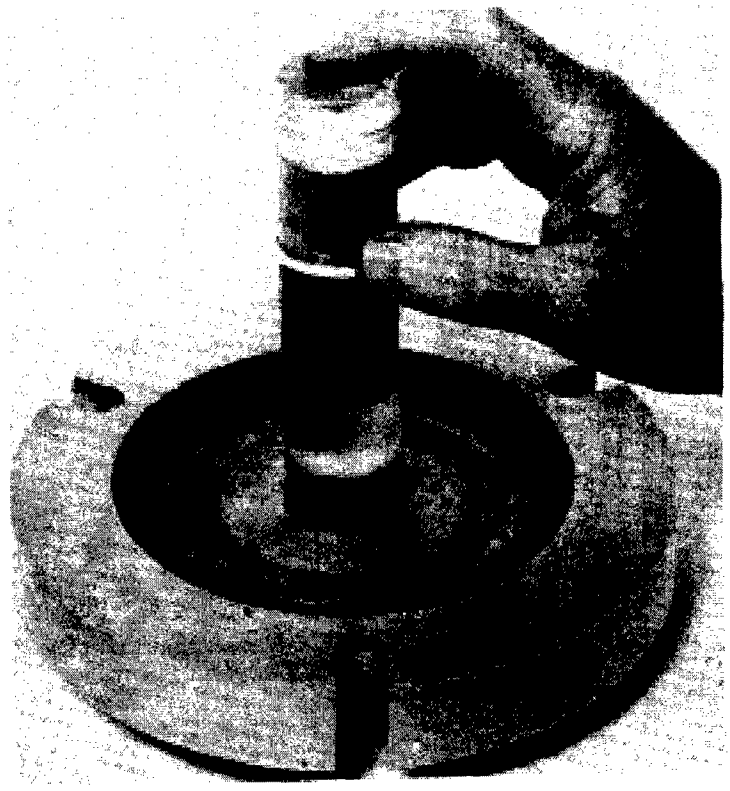


Figure 10. Rolling rubber membrane over a 1.4-in.-diameter specimen

(4) Assemble the triaxial chamber and place it in position in the loading device. Connect the tube from the pressure reservoir to the base of the triaxial chamber. With valve C (see Figure 11) on the pressure reservoir closed and valves A and B open, increase the pressure inside the reservoir and allow the pressure fluid to fill the triaxial chamber. Allow a few drops of the pressure fluid to escape through the vent valve (valve B) to insure complete filling of the chamber with fluid. Close valve A and the vent valve.

(5) With valves A and C closed, adjust the pressure regulator to preset the desired chamber pressure. The range of chamber pressures for the three specimens will depend on the loadings expected

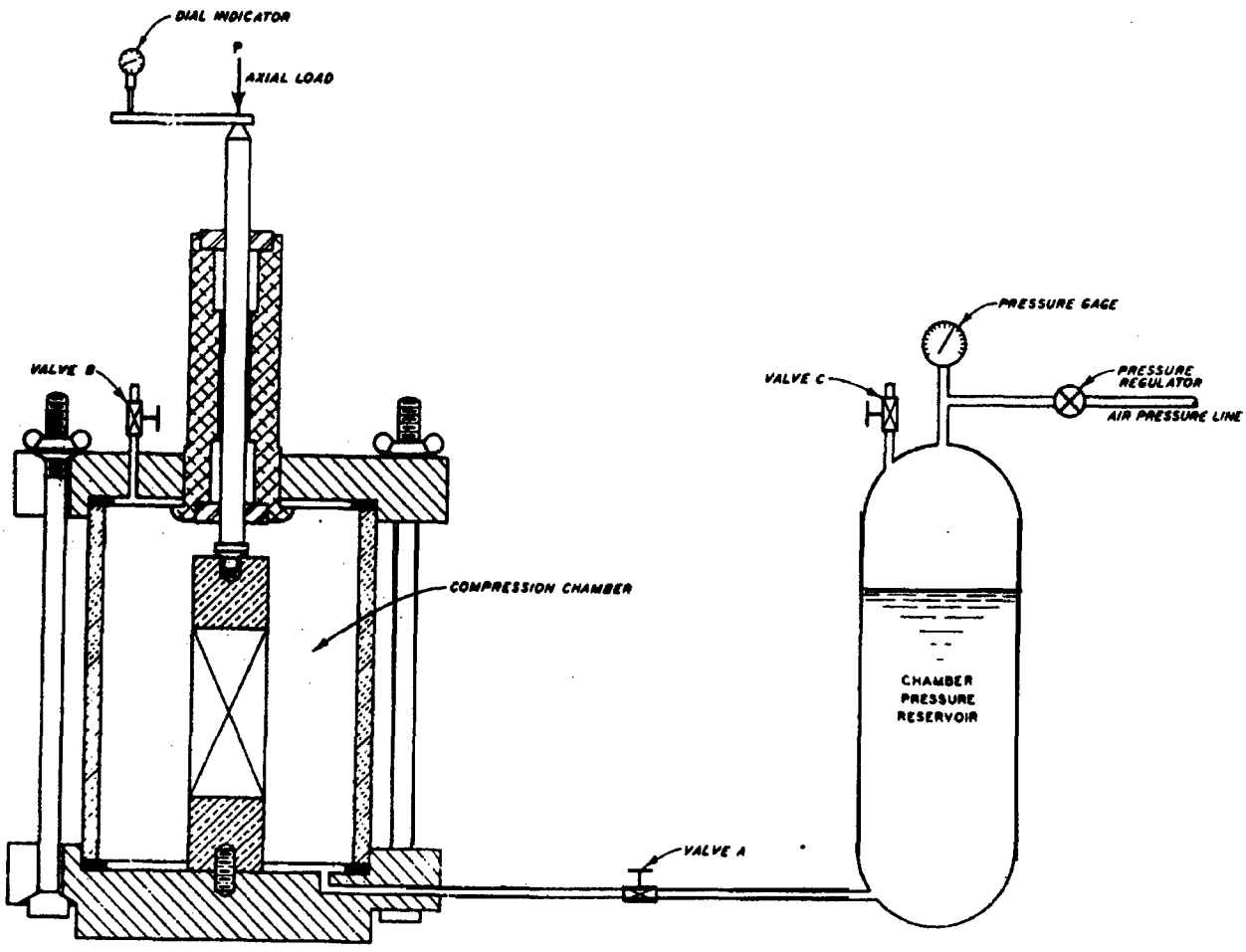


Figure 11. Schematic diagram of triaxial compression apparatus for Q test

in the field. The maximum confining pressure should be at least equal to the maximum normal load expected in the field in order that the shear strength data need not be extrapolated for use in design analysis. Record the chamber pressure on data sheets (Plates X-1 and X-2). Now open valve A and apply the preset pressure to the chamber. Application of the chamber pressure will force the piston upward into contact with the ram of the loading device. This upward force is equal to the chamber pressure acting on the cross-sectional area of the piston minus the weight of the piston minus piston friction.

(6) Start the test with the piston approximately 0.1 in. above the specimen cap. This allows compensation for the effects of piston friction, exclusive of that which may later develop as a result of lateral forces. Set the load indicator to zero when the piston comes into contact with the specimen cap. In this rnanner the upward thrust of the chamber pressure on the piston is also eliminated from further consideration. Contact of the piston with the specimen cap is indicated by a slight movement of the load indicator. Set the strain indicator and record on the data sheet (Plate X-2) the initial dial reading at contact. Axially strain the specimen at a rate of about 1 percent per minute (for plastic materials) and about 0.3 percent per minute (for brittle materials that achieve maximum deviator stress at about 3 to 6 percent strain); at these rates the elapsed time to reach maximum deviator stress would be about 15 to 20 min.

(7) Observe and record the resulting load at every 0.3 percent strain for about the first 3 percent and, thereafter, at every 1 percent, or for large strains, at every 2 percent strain; sufficient readings should be taken to completely define the shape of the stress-strain curve so frequent readings may be necessary as failure is approached. Continue the test until an axial strain of 15 percent has been reached, as shown in Figures 12a, 12b, and 12d; however, when the deviator stress decreases after attaining a maximum value and is continuing to decrease at 15 percent strain (Fig. 12c), the test shall be continued to 20 percent strain.

(8) For brittle soils (i.e., those in which maximum deviator stress is reached at 6 percent axial strain or less), tests should be performed at rates of strain sufficient to produce times to failure as set forth in paragraph 5a(6) above; however, when the maximum deviator stress has been clearly defined, the rate may be increased such that the remainder of the test is completed in the same length of time as that taken to reach maximum deviator stress. However, for each group of tests about 20 percent of the samples should be tested at the rates set forth

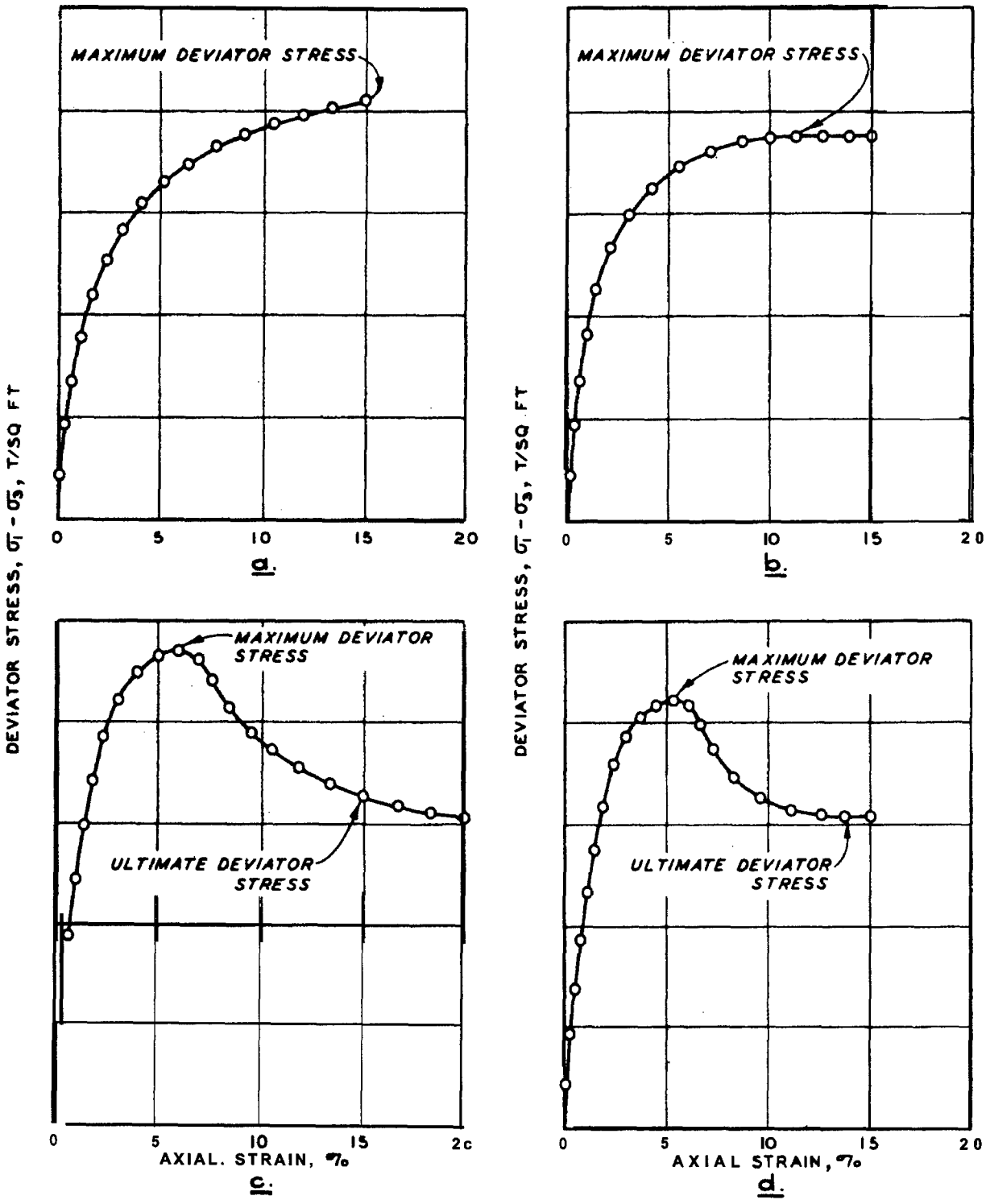


Figure 12. Examples of stress-strain curves

in paragraphs 5a(6) and 5a(7) above.

(9) Upon completion of axial loading, release the chamber pressure by shutting off the air supply with the regulator and opening valve C. Open valve B and draw the pressure fluid back into the pressure reservoir by applying a low vacuum at valve C. Dismantle the triaxial chamber. Make a sketch of the specimen, showing the mode of failure.

(10) Remove the membrane from the specimen For 1.4-in.-diameter specimens, carefully blot any excess moisture from the surface of the specimen and determine the water content of the whole

specimen (see Appendix I, WATER CONTENT - GENERAL). For 2.8-in.-diameter or larger specimens, it is permissible to use a representative portion of the specimen for the water content determination. It is essential that the final water content be determined accurately, and weighings should be verified, preferably by a different technician.

(11) Repeat the test on the two remaining specimens at different chamber pressures, though using the same rate of strain.

b. Computations. The computations shall consist of the following steps:

(1) From the observed data, compute and record on the data sheet (Plate X-1) the initial water content (see Appendix I, WATER CONTENT - GENERAL), volume of solids, initial void ratio, initial degree of saturation, and initial dry density, using the formulas given in Appendix II, UNIT WEIGHTS, VOID RATIO, POROSITY, AND DEGREE OF SATURATION.

(2) Compute and record on the data sheet (Plate X-2) the axial strain, the corrected area, and the deviator stress at each increment of strain, using the following formulas:

$$\text{Axial strain, } \varepsilon = \frac{\Delta H}{H_o}$$

$$\text{Corrected area of specimen, } A_{corr}, \text{ sq cm} = \frac{A_o}{2 - \varepsilon}$$

$$\text{Deviator stress, tons per sq ft} = \frac{P}{A_{corr}} \times 0.465$$

where ΔH = change in height of specimen during test, cm

H_o = initial height of specimen, cm, (Where a significant decrease in specimen volume occurs upon application of the chamber pressure, as in partially saturated soils, the height of the specimen after application of the chamber pressure should be used rather than the initial height.)

A_o = initial area of specimen, sq cm

P = net applied axial load, lb (the actual load applied to specimen after correction for piston friction and for the upward thrust of the fluid pressure in the triaxial chamber)

(3) Record the time to failure on the data sheet (Plate X-2).

(4) The rubber membrane increases the apparent strength of the specimen. Investigations† with specimens 1.5 in. in diameter and membranes 0.008 in. thick, for instance, indicate the increase in deviator stress to be 0.6 psi at 15 percent axial strain. The correction, σ_r, to be made to the measured deviator stress for the effect of the rubber membrane is computed as follows:

$$\sigma_r = \frac{\pi D_o M \varepsilon(1 - \varepsilon)}{A_o}$$

where D_o = initial diameter of specimen
M = compression modulus of the rubber membrane
ε = axial strain
A_o = initial cross-sectional area of the specimen

The compression modulus may, without great error, be assumed to be equal to that measured in extension. An apparatus for determining the extension modulus of rubber is described in another work.† In tests of very soft soils the membrane effect may be significant, and in these tests it is advisable to compute or estimate the correction and deduct it from the maximum deviator stress. For most soils tested using membranes of standard thickness, the correction is insignificant and can be ignored.

c. Presentation of Results. The results of the Q test shall be recorded on the report form shown as Plate X-3. Enter pertinent information regarding the condition of the specimen or method of preparing the

† Bishop and Henkel, op. cit., pp. 167-171.

specimen under "Remarks." Plot the deviator stress versus the axial strain for each of the specimens as shown in Figure 12. The peak or maximum deviator stress represents "failure" of the specimen; when the deviator stress increases continuously during the test, the deviator stress at 15 percent axial strain shall be considered the maximum deviator stress. When the deviator stress decreases after reaching a maximum, the minimum deviator stress attained before 15 percent axial strain shall be considered the ultimate deviator stress, as shown in Figures 12c and 12d. Construct Mohr stress circles on an arithmetic plot with shear stresses as ordinates and normal stresses as abscissas. As shown in Figure 13, the applied principal stresses, σ_1 and σ_3, are plotted on the abscissa, and the Mohr circles are drawn with radii of one-half the maximum deviator stresses $\left(\frac{\sigma_1 - \sigma_3}{2}\right)$ and with their centers at values equal to one-half the sums of the major and minor principal stresses $\left(\frac{\sigma_1 + \sigma_3}{2}\right)$. Plot a Mohr circle, or a sufficient segment thereof, for each specimen in

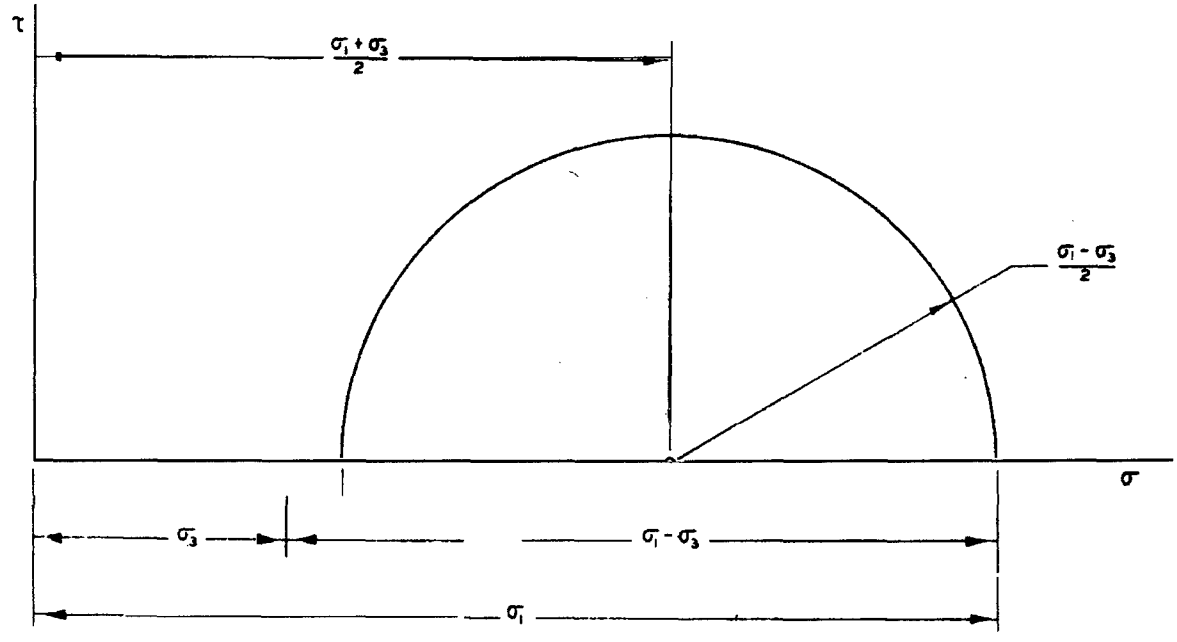


Figure 13. Construction of Mohr's circle of stress

the graph in the upper right corner of report form. A sketch of each specimen after failure should be shown above the Mohr circles (Plate X- 3). The following procedures should be followed in drawing strength envelopes:

(1) Undisturbed specimens. For undisturbed specimens, the strength envelope should be drawn tangent to the Mohr circles. Q tests of saturated soils usually indicate a strength envelope that is parallel to the abscissa as shown in Figure 14a, so the angle of internal friction is usually equal to zero. Strength envelopes indicated by Q tests on partially saturated soils are usually curved as shown in Figure 14b, particularly for the lower normal stresses. When the curvature is pronounced, the shear strength parameters ϕ and c are not constants.

(2) Compacted specimens. For compacted specimens, the strength envelope should be drawn through points on the Mohr circles representing stresses on the failure plane as shown in Figure 14c.

6. Q TEST WITH BACK-PRESSURE SATURATION. In cases where a foundation'soil exists that is partially saturated during exploration but which will become completely saturated without significant volume change either before or during construction, it is necessary to saturate Q test specimens, using back pressure, before they are sheared. Such field conditions may occur when, due to heavy rains or other reasons, the water table is raised above the level that existed during initial sampling: Construction of cofferdams, river diversions, and closure sections, or percolation of rainwaters can also create conditions that increase foundation water contents after exploration but before embankment construction and subsequent consolidation of the foundation.

For the Q test with back-pressure saturation, the apparatus should be set up similar to that shown in Figure 16 (page X-30). Filter strips should not be used and as little volume change as possible should be permitted during the test. After completing the steps outlined in paragraphs 4 and 5a(1) through 5a(4) (note that the procedures for attaching the membrane to the cap and base and for deairing the drainage lines are similar

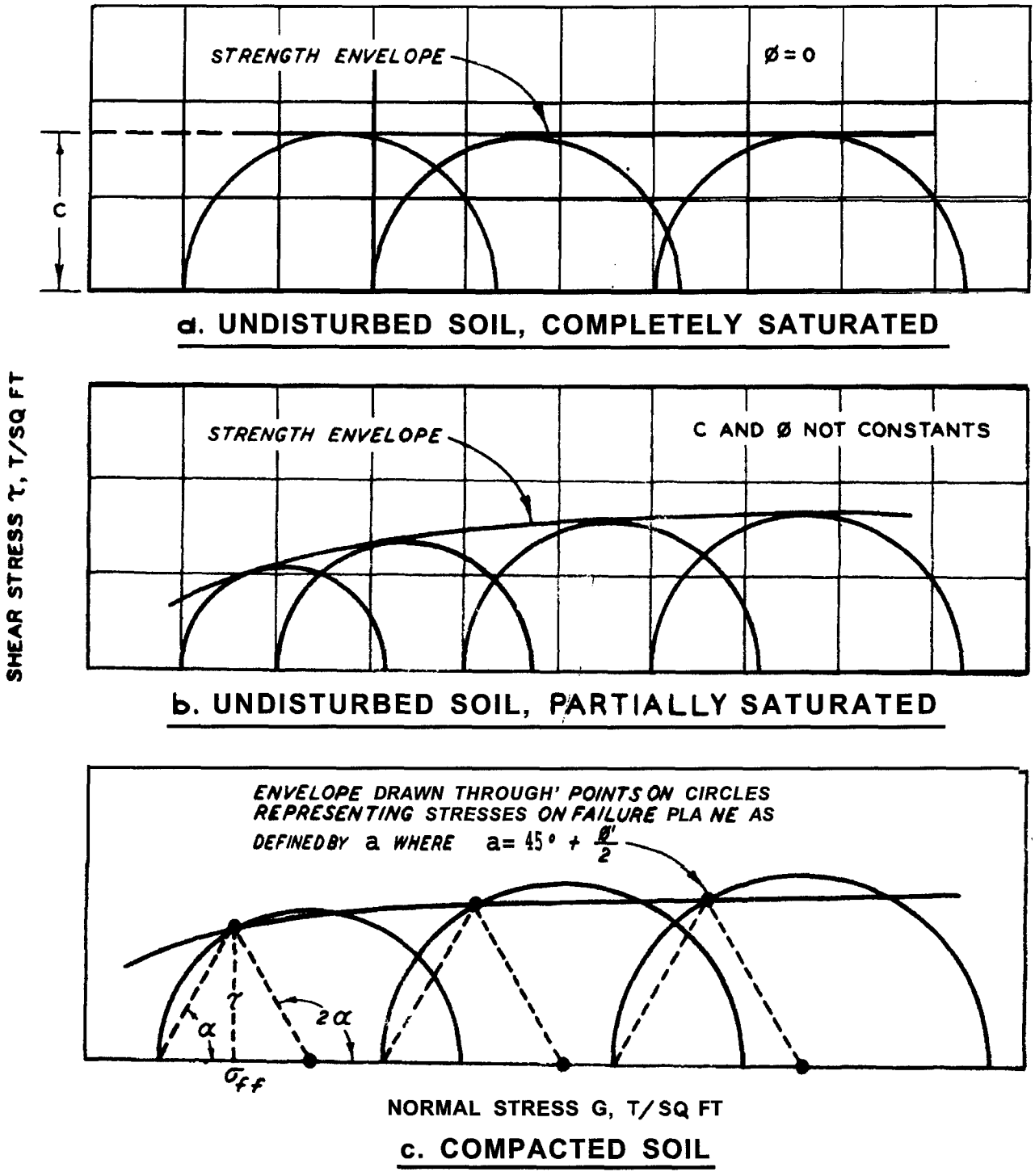


Figure 14. Examples of strength envelopes for Q tests

to those used in the R test), apply 3-psi chamber pressure to the specimen with all drainage valves closed. Allow a minimum of 30 min for stabilization of the specimen pore water pressure, measure AH, and begin back-pressure procedures as given in paragraphs 7b(2) through 7b(5). After verification of saturation, and remeasurement of AH, close all drainage lines leading to the back pressure and pore water measurement apparatus. Holding the maximum applied back pressure constant, increase the chamber pressure until the difference 'between the chamber pressure and the back pressure equals the desired effective confining pressure (see paragraph 5a(5)). Then proceed as outlined in paragraphs 5a(6) through 5a(11).

7. R TEST. All specimens must be completely saturated before application of the deviator stress in the R test. A degree of saturation over 98 percent can be considered to represent a condition of essentially complete saturation; if pore water pressures are to be measured during shear, however, the specimens must be 100 percent saturated. Computations of

the degree of saturation based on changes of volume and water content are often imprecise, so complete saturation of a specimen should be assumed only when an increase of the chamber fluid pressure will cause an immediate and equal increase of pressure in the pore water of the specimen. In general, it is preferable to saturate the soil after the specimens have been prepared, encased in membranes, and placed within the compression chamber, using back pressure. A back pressure is an artificial increase of the pore water pressure which will increase the degree of saturation of a specimen by forcing pockets of air into solution in the pore water. The back pressure is applied to the pore water simultaneously with an equal increase of the chamber pressure so that the effective stress acting on the soil skeleton is not changed. In other words, the pressure differential across the membrane remains constant during the back pressure saturation phase. Thus, when the back pressure is increased sufficiently slowly to avoid an excessive pressure differential within the specimen itself, the degree of saturation will be increased while the volume of the specimen is maintained essentially constant. Figure 15 gives the back pressure theoretically required to produce a desired increase in saturation if there is no change in specimen volume. It is important to note that the relation shown in Figure 15 is based on an assumption that the water entering the specimen contains no dissolved air.

a. Apparatus. In addition to the apparatus described in paragraphs 3a through 3g, the following equipment are necessary for R tests utilizing back pressure for saturation:

(1) Air reservoir and regulator for controlling the back pressure, similar to those used to control the chamber pressure.

(2) Bourdon gage attached to the back pressure reservoir to measure the applied back pressure. As relatively large back pressures and chamber pressures are sometimes required, it is essential that these two pressures be measured accurately to insure that the precise

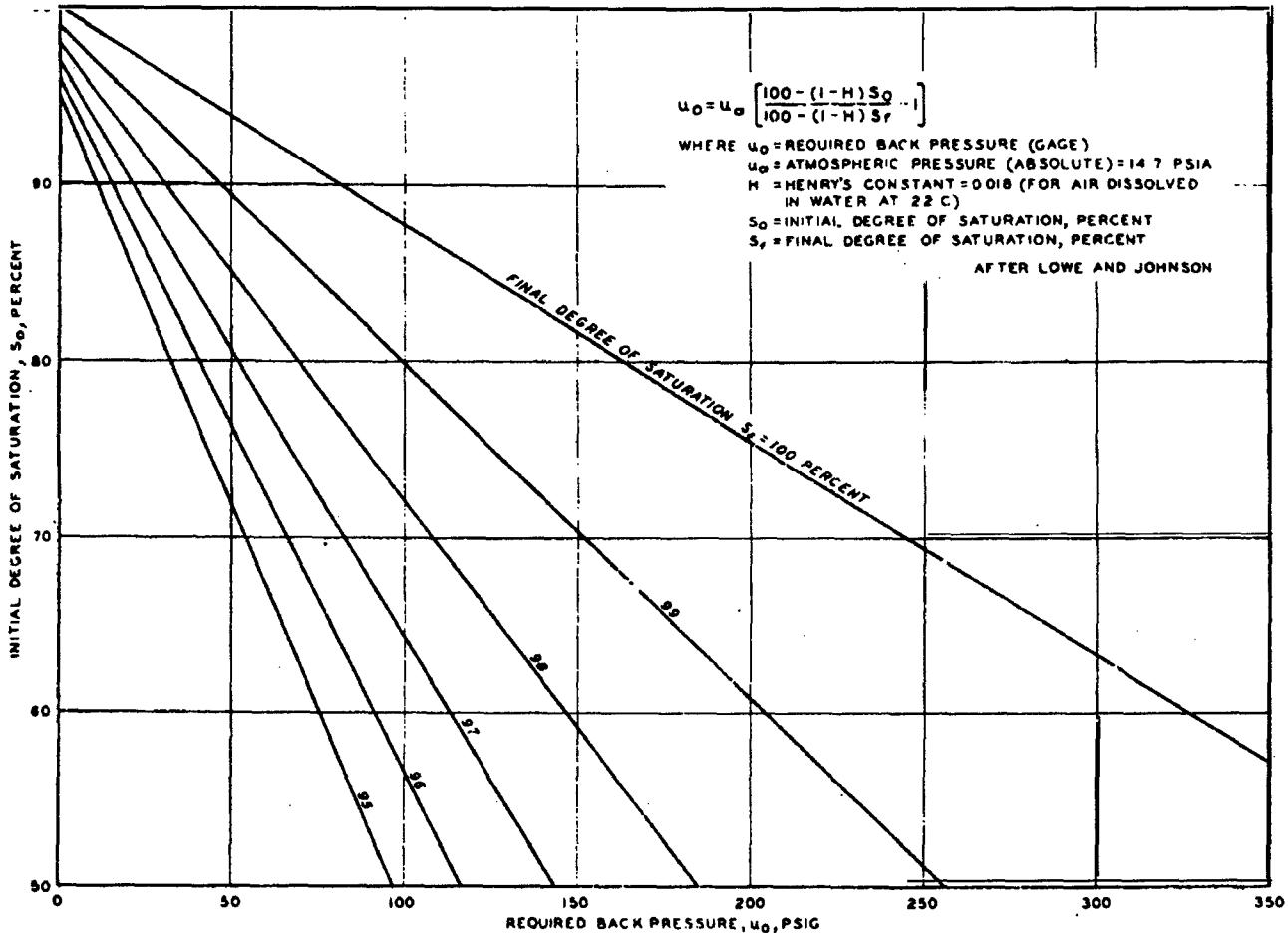


Figure 15. Back pressure required to attain various degrees of saturation

difference between them is known. A differential pressure gage† will permit this difference to be measured directly.

(3) Calibrated burette or standpipe capable of measuring volume changes to within 0.1 cc for 1.4-in.-diameter specimens, 0.5 cc for 2.8-in.-diameter specimens, and 1 cc for 6-in.-diameter specimens. This burette is connected in the back pressure line leading to the top of

† John Lowe, III, and Thaddeus C. Johnson, "Use of back pressure to increase degree of saturation of triaxial test specimens," ASCE Research Conference on Shear Strength of Cohesive Soils, University of Colorado (Boulder, Colo., June 1960).

the specimen to measure the volume of water added to the specimen during saturation and volume changes of the specimen during consolidation. If the water added to the specimen becomes saturated with air, a higher back pressure will be required than that given in Figure 15. Therefore, precautions should be taken to minimize aeration of the saturation water by reducing the area of the air-water interface or by separating the air and water with a rolling rubber diaphragm.* A relatively long (over 6-foot) length of thick-walled, small-bore tubing between the burette and the specimen will also reduce the amount of air entering the specimen. Adequate safety precuations should be taken against breakage of the burette under high pressures.

(4) Electrical pressure transducer or no-flow indicator with which the pressure of the pore water at the bottom of the specimen can be measured without allowing a significant flow of water from the specimen. This is an extremely difficult measurement to make since even a minute flow of water will reduce the pressure in the pore water; yet the measuring device must be sensitive enough to detect small changes in pressure. Electrical pressure transducers, while relatively expensive, offer almost complete protection against flow, are simple to operate, and lend themselves to the automatic recording of test data. Several types of manually balanced pressure-measuring systems employing a no-flow indicator are being used successfully,‡ though a full discussion of their relative merits

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* H. B. Seed, J. K. Mitchell, and C. K. Chan, "The strength of compacted cohesive soils," ASCE Research Conference on Shear Strength of Cohesive Soils, University of Colorado (Boulder, Colo., June 1960).

‡ Bishop and Henkel, op. cit., pp. 52-63, 206-207.

A. Andersen, L. Bjerrum, E. DiBiagio, and B. Kjaernsli, Triaxial Equipment Developed at the Norwegian Geotechnical Institute, Publication No. 21, Norwegian Geotechnical Institute (Oslo, 1957).

A. Casagrande and R.C. Hirschfeld, First Progress Report on Investigation of Stress-Deformation and Strength Characteristics of Compacted Clays, Soil Mechanics Series No. 61, Harvard University (Cambridge, Mass., May 1960).

and shortcomings is not possible here.

b. Procedure. The procedure for the R test utilizing back pressure for saturation shall consist of the following steps:

(1) Proceed as outlined in paragraphs 5a(1) through 5a(4), with the exception that specimen bases and caps with porous inserts and drainage connections should be used and back pressure equipment should be included as shown in Figure 16. Saturated strips of filter paper (such as Whatman's No. 54) placed beneath the membrane and extending from the base along three-fourths of the specimen length will reduce the time required for saturation and consolidation. These strips must neither overlap and form a continuous circumferential coverage of the specimen

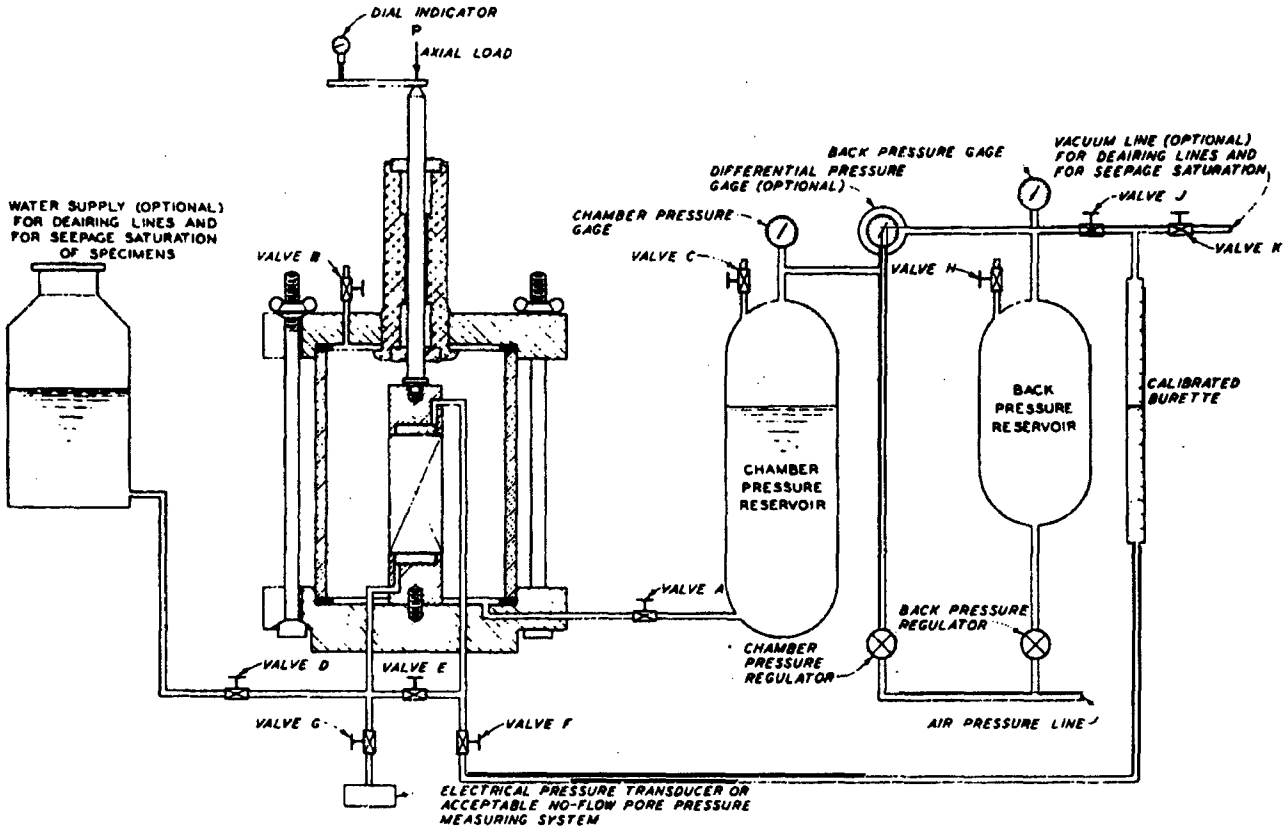


Figure 16. Schematic diagram of typical triaxial compression apparatus for R and S tests

nor form a continuous path between the base and the cap. Place saturated filter paper disks having the same diameter as that of the specimen between the specimen and the base and cap; these disks will also facilitate removal of the specimen after the test. The drainage lines and the porous inserts should be completely saturated with deaired water. The drainage lines should be as short as possible and made of thick-walled, small-bore tubing to insure minimum elastic changes in volume due to changes in pressure. Valves in the drainage lines (valves E, F, and G in Figure 16) should preferably be of a type which will cause no discernible change of internal volume when operated (such as the Teflon-packed ball valve made by the Whitey Research Tool Co.). While mounting the specimen in the compression chamber, care should be exercised to avoid entrapping any air beneath the membrane or between the specimen and the base and cap.

(2) Estimate the magnitude of the required back pressure by reference to Figure 15 or other theoretical relations. Specimens should be completely saturated before any appreciable consolidation is permitted, for ease and uniformity of saturation as well as to allow volume changes during consolidation to be measured with the burette; therefore, the difference between the chamber pressure and the back pressure should not exceed 5 psi during the saturation phase. To insure that a specimen is not prestressed during the saturation phase, the back pressure must be applied in small increments, with adequate time between increments to permit equalization of pore water pressure throughout the specimen.

(3) With all valves closed, adjust the pressure regulators to a chamber pressure of about 7 psi and a back pressure of about 2 psi. Record these pressures on the data sheet (Plate X-4). Now open valve A to apply the preset pressure to the chamber fluid and simultaneously open valve F to apply the back pressure through the specimen cap. Immediately open valve G and read and record the pore pressure at the specimen base. When the measured pore pressure becomes essentially

constant, close valves F and G† and record the burette reading.

(4) Using the technique described in step (3), increase the chamber pressure and the back pressure in increments, maintaining the back pressure at about 5 psi less than the chamber pressure. The size of each increment might be 5, 10, or even 20 psi, depending on the compressibility of the soil specimen and the magnitude of the desired consolidation pressure. Open valve G and measure the pore pressure at the base immediately upon application of each increment of back pressure and observe the pore pressure until it becomes essentially constant. The time required for stabilization of the pore pressure may range from a few minutes to several hours depending on the permeability of the soil. Continue adding increments of chamber pressure and back pressure until, under any increment, the pore pressure reading equals the applied back pressure immediately upon opening valve G.

(5) Verify the completeness of saturation by closing valve F and increasing the chamber pressure by about 5 psi. The specimen shall not be considered completely saturated unless the increase in pore pressure immediately equals the increase in chamber pressure.

(6) When the specimen is completely saturated, hold the maximum applied back pressure constant and increase the chamber pressure until the difference between the chamber pressure and the back pressure equals the desired consolidation pressure. Open valve F and permit the specimen to consolidate (or swell.) under the consolidation pressure. Valve E may be opened to allow drainage from both ends of the specimen. At increasing intervals of elapsed time (0.1, 0.2, 0.5, 1, 2, 4, 8, 15, and 30 min, 1, 2, 4, and 8 hr, etc.), observe and record (Plate X-5) the burette readings and, if practicable, the dial indicator readings (it may be necessary to force the piston down into contact with the specimen cap for each

† If an electrical pressure transducer is used to measure the pore pressure, valve G may be safely left open during the entire saturation procedure.

reading). Plot the burette readings (and dial indicator readings, if taken) versus the logarithm of elapsed time, as shown in Figure 5 of Appendix VIII, CONSOLIDATION TEST. Allow consolidation to continue until a marked reduction in slope of the curve shows that 100 percent primary consolidation has been achieved.

(7) Close valve G, unless pore pressure measurements are to be made during shear, and valves E and F, and proceed according to paragraphs 5a(6) through 5a(10), except use a rate of strain for the R test of about 0.5 percent per minute (for plastic materials) and about 0.3 percent per minute or less for brittle materials that achieve a maximum deviator stress at about 3 to 6 percent strain; the strain rate used should result in a time to maximum deviator stress of approximately 30 min. Relatively pervious soils may be sheared in 15 min. These rates of strain do not permit equalization of induced pore pressure throughout the specimen and are too high to allow satisfactory pore pressure measurements to be made at the specimen ends during shear.† Therefore, these rates of strain are applicable only to R tests in which no pore pressure measurements are made during shear. Where pore pressure measurements are made at the ends of the specimens as in $\bar{R}$ tests, the time to reach maximum deviator stress should generally be at least 120 min; considerably longer time may be required for materials of low permeability. For brittle soils (i.e., those in which the maximum deviator stress is reached at 6 percent axial strain or less), after the maximum deviator stress has been clearly defined, the rate of strain may be increased so that the remainder of the test is completed in the same length of time as that taken to reach maximum deviator stress. However, for each group of tests in a given test program, at least 20 percent of the samples should be tested to final axial strain at rates of strain outlined in the first sentence of this paragraph.

c. Computations. The computations shall consist of the following steps:

(1) From the observed data, compute and record on the data sheet (Plate X-1) the initial water content, volume of solids, initial void

† Bishop and Henkel, op. cit., pp. 192-204.

ratio, initial degree of saturation, and initial dry density, using the formulas previously presented.

(2) Compute the cross-sectional area of the specimen after completion of consolidation according to the formula:†

$$\text{Area of specimen after consolidation, } A_c, \text{ sq cm} = A_o \frac{H_o - 2\Delta H_o}{H_o}$$

or if the specimen is or has been completely saturated during the test, use the more accurate formula:

$$\text{Area of specimen after consolidation, } A_c, \text{ sq cm} = \frac{V_o - V_a - \Delta V_w}{H_o - \Delta H_o}$$

where V_o = initial volume of specimen, cc

V_a = initial volume of air in specimen, cc = $V_o - V_s - V_w$

$V_o - V_s - V_w$ = initial volume of specimen minus volume of solids minus initial volume of water

ΔV_w = change in volume of water in the specimen during the saturation and consolidation phases of the test, cc. This value may be computed from the change in weight of the specimen before and after the test or from the burette readings from the start of saturation on to the end of consolidation

H_o = initial height of specimen, cm

ΔH_o = change in height of specimen during consolidation, cm

(3) Using the computed dimensions of the specimen after consolidation and assuming that the water content after consolidation is the same as the final water content, compute the void ratio and degree of saturation using formulas previously presented.

(4) Compute and record on the data sheet (Plate X-2) the axial strain, the corrected area, and the deviator stress at each increment of strain, using the following formulas:

† This formula is based on the assumption that axial and radial strains are equal during consolidation.

$$\text{Axial strain, } \varepsilon = \frac{\Delta H}{H_c}$$

$$\text{Corrected area of specimen, } A_{corr}\text{, sq cm} = \frac{A_c}{1 - \varepsilon}$$

$$\text{Deviator stress, tons per sq ft} = \frac{P}{A_{corr}} \times 0.465$$

where H_c = height of specimen after consolidation, cm = $H_o - \Delta H_o$

P = net applied axial load, lb (see paragraph 5b(2))

(5) Record the time to failure on the data sheet (Plate X-2).

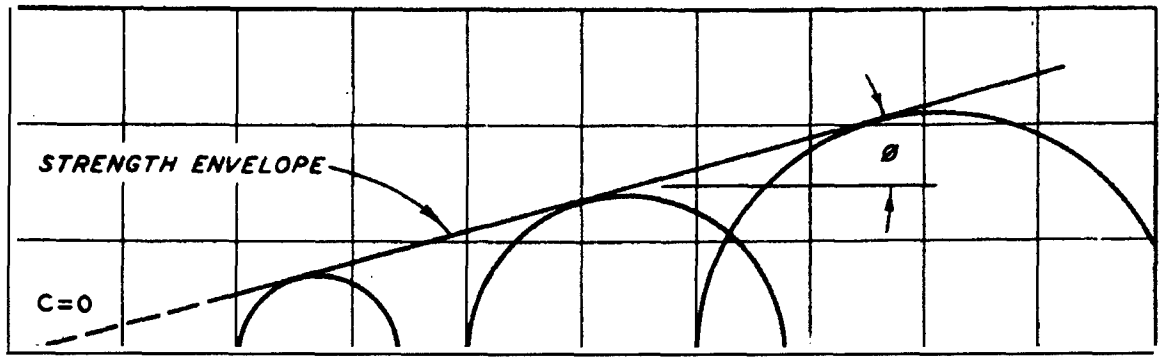
(6) Correct the maximum deviator stress, if necessary, for the effect of membrane restraint (see paragraph 5b(4)).

d. Presentation of Results. The results of the R test shall be presented on the report form shown as Plate X-3, as described in paragraph 5c. A sketch of each specimen after failure should be shown above the Mohr circles. If pore pressure measurements were made during shear, plot the induced pore pressure versus axial strain for each specimen below the stress-strain curves. The procedures below should be followed in drawing strength envelopes:

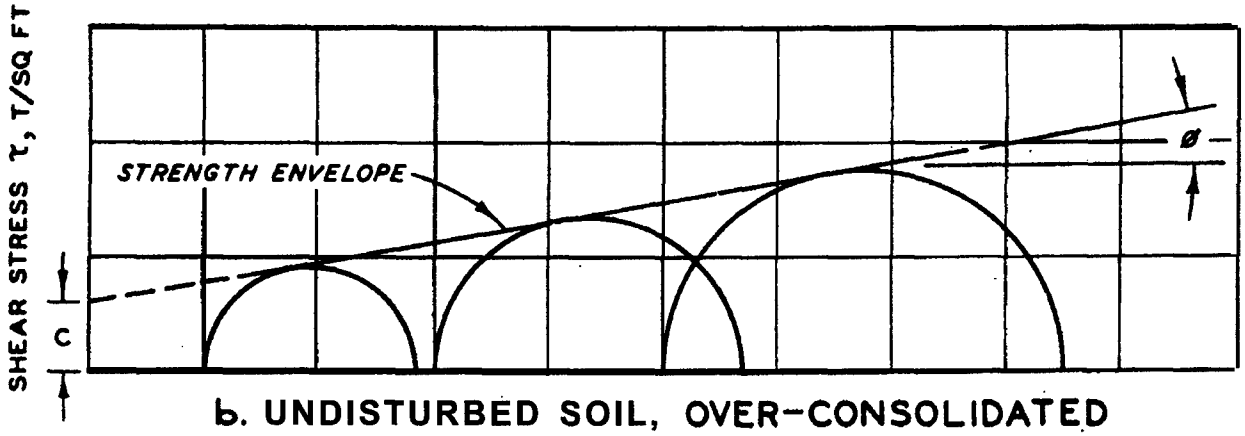
(1) Undisturbed specimens. For undisturbed specimens, strength envelopes should be drawn tangent to the Mohr circles as shown in Figures 17a and 17b.

(2) Compacted specimens. For compacted specimens, strength envelopes should be drawn through points on the Mohr circles representing stresses on the failure plane as shown in Figure 17c.

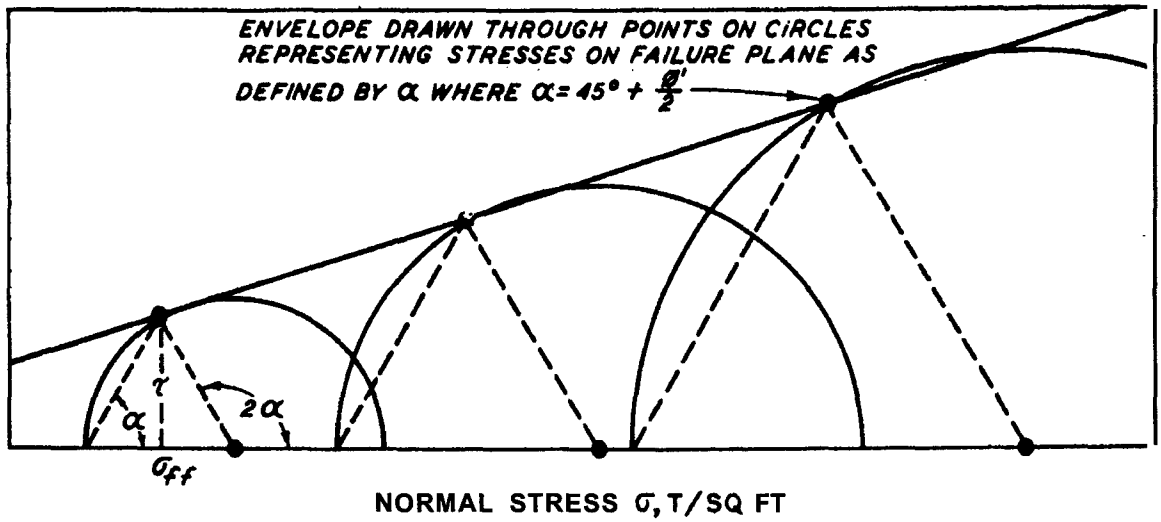
8. S TEST. The S test using triaxial equipment, as a rule, shall be performed only with relatively pervious soils. The consolidation of triaxial specimens of relatively impervious soils proceeds so slowly that the time required to complete an S triaxial test inhibits its use in



a. UNDISTURBED SOIL, NORMALLY CONSOLIDATED



b. UNDISTURBED SOIL, OVER-CONSOLIDATED



c. COMPACTED SOIL

Figure 17. Examples of strength envelopes for R tests

routine laboratory work. Therefore, S tests of fine-grained impervious materials should normally be performed with direct shear equipment (see Appendix IX DRAINED (S) DIRECT SHEAR TEST). However, if scheduling permits, it may be desirable to perform companion S triaxial tests of impervious soils to compare the results with those from S direct shear tests. If the soil to be tested is relatively impervious and contains gravel which would preclude the use of direct shear equipment, consideration should. be given to using triaxial equipment with pore pressure measurements in order to obtain the drained shear strength parameters within a reasonable length of time. All specimens must be completely saturated before application of the deviator stress in the S test.

a. Apparatus. The apparatus used to perform the R test, as illustrated in Figure 16, will usually be satisfactory for the S test, though the equipment for saturation by back pressure will not be necessary for relatively pervious soils. In general, controlled-strain testing should be used for relatively pervious soils, and controlled-stress testing should be used for relatively impervious soils.

b. Procedure. The procedure for the S test shall consist of the following steps:

(1) For soils requiring saturation by back pressure, proceed as outlined in paragraphs 6b(1) through 6b(6). For pervious soils which can be effectively saturated by seepage, that is, by water percolating through the specimen under a small hydraulic head, omit the back pressure equipment described in paragraph 6a and proceed as follows:

(a) Proceed as outlined in paragraphs 5a(1) through 5a(4), with the exception that specimen bases and caps with porous inserts and drainage connections should be used and the apparatus should include a water supply container and a calibrated burette with a vacuum connection as shown in Figure 16. For specimens of cohesionless soil prepared as described in paragraph 4b, the porous inserts and drainage lines (including the burette) should be dry, and a low vacuum (less than 5 psi) should be

maintained at both the specimen base and cap (with valves D and G closed) to support the specimen while assembling and filling the triaxial chamber.

(b) Keeping valves A and C closed, adjust the pressure regulator for a chamber pressure of about 5 psi and then open valve A to apply this pressure to the chamber.

(c) With valves E and G closed, maintain a low vacuum through the burette to the specimen cap. Then open valve D and elevate the water supply container so that a hydrostatic head of 1 to 2 ft is applied to the base of the specimen.

(d) When the saturation water rises into the burette, disconnect the vacuum from the burette. Permit seepage under the small head to continue until the rate of flow into the burette is constant, and then close valve D.

(2) With valves E and F open (see Fig. 16), lower the piston into contact with the specimen cap and increase the axial load at a relatively slow rate so that a fully drained condition exists at failure with controlled-strain loading or after each increment of load with controlled-stress application. As for the direct shear test, considerable experience and judgment are generally required in determining the proper rate of axial load application (see Appendix IX, DRAINED (S) DIRECT SHEAR TEST). Theoretical formulas are also available† for estimating the time required for failure in S tests. Special precautions may be necessary for tests requiring an axial loading duration in excess of a few hours to insure that the chamber pressure (as well as the back pressure, if used) is maintained constant, that temperature fluctuations are minimized, and that evaporation or aeration of the water in the burette is reduced as much as possible. Placing about 1 cc of oil or dyed kerosene over the water surface in the burette will minimize evaporation.

(3) Record the dial indicator and burette readings at

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† Bishop and Henkel, op. cit., pp. 124-127, 204-206.

increasing intervals of elapsed time under each increment of load. For relatively impervious soils, plot either or both of these readings versus the logarithm of elapsed time, as shown in Figure 5 of Appendix VIII, CONSOLIDATION TEST, to establish when primary consolidation has been essentially completed for each increment of load. Record the final dial indicator and burette readings for each axial load increment on a form similar to Plate X-6 prior to applying the next increment. With controlled-strain loading, periodically observe and record (Plate X-6) the resulting load and the dial indicator and burette readings; sufficient readings should be taken to completely define the shape of the stress-strain curve, Continue the test until an axial strain of 15 percent has been reached; however, when the deviator stress decreases after attaining a maximum value and is continuing to decrease at 15 percent strain, the test shall be continued to 20 percent strain (see Fig. 12).

(4) Upon completion of axial loading, close valves E and F and proceed as outlined in paragraphs 5a(8) through 5a(10), except measure the specimen diameter, as described in paragraph 4a(1)(e), after the compression chamber has been dismantled. While considerable difficulty may be encountered in measuring the diameter of the specimen after the test, such measurements will permit the most reliable computations of the specimen properties at failure.

c. Computations. The computations shall consist of the following steps:

(1) From the observed data, compute and record on the data sheet (Plate X-1) the initial water content, volume of soilds, initial void ratio, initial degree of saturation, and initial dry density using the formulas previously presented.

(2) Compute the cross-sectional area of the specimen, A_c, after completion of consolidation using the formulas presented in paragraph 6c(2).

(3) Using the dimensions of the specimen after consolidation

and the changes in volume as measured with the burette, compute the void ratio and degree of saturation after consolidation using the formulas previously presented.

(4) Compute and record on the data sheet (Plate X-6) the axial strain, the corrected area, and the deviator stress corresponding to the final readings under each increment of load for controlled-stress loading or for convenient intervals of strain for controlled-strain loading using the following formulas:

$$\text{Axial strain, } \varepsilon = \frac{\Delta H}{H_c}$$

$$\text{Area of specimen corrected for strain and volume change, } A'_{corr}, \text{ sq cm} = \frac{A_c}{1 - C\varepsilon}$$

$$\text{Deviator stress, tons per sq ft} = \frac{P}{A'_{corr}} \times 0.465$$

where C = correction for volume change during shear $= \frac{A_f}{A_e}$

A_f = area of specimen after test based on measurements

$$= \frac{\pi D_f^2}{4} = 0.7854\ D_f^2$$

A_e = area of specimen at end of test computed on basis of constant volume

$$= \frac{A_c}{1 - \varepsilon_e}$$

ε_e = axial strain at end of test $= \frac{H_c - H_f}{H_c}$

P = net applied axial load, lb (see paragraph 5b(2))

(5) Record the time to failure on the data sheet (Plate X-6).

(6) Correct the maximum deviator stress, if necessary, for the effect of membrane restraint (see paragraph 5b(4)).

d. Presentation of Results. The results of the S test shall be presented on the report form shown as Plate X-3, as described in paragraph 5c. If volume changes of the specimens during shear were measured, plot the volumetric strain versus axial strain for each specimen below the stress-strain curves.

9. POSSIBLE ERRORS. Following are possible errors that would cause inaccurate determinations of strength and stress -deformation characteristics :

a. Apparatus. (1) Leakage of chamber fluid into specimen. Such leakage might occur through or around the ends of the membrane or through the drainage connections and it would decrease the effective stress in a specimen during undrained shear, Very little leakage is needed to cause a very large change in effective stress, and the longer the period of undrained shear, the greater the amount of leakage. (Leakage will not influence the effective stress during periods of specimen drainage, but it will introduce errors in volume change measurements,)

(2) Leakage of pore water out of specimen. This leakage might occur through fittings or valves and it would increase the effective stress in a specimen during undrained shear.

(3) Permeability of porous inserts too low.

(4) Restraint caused by membrane and filter paper strips.

(5) Piston friction.

b. Preparation of Specimens. (1) Specimen disturbed while trimming. Disturbance of the natural soil structure does not always result in strength measurements which are too low, that is, on the safe side; disturbed specimens will consolidate more under the effective consolidation pressure in R or S tests and the measured strengths will be too high.

(2) Specimen disturbed while enclosing with membrane. The techniques of placing the membrane around the specimen illustrated in Figures 9 and 10 may not be satisfactory for sensitive undisturbed soils since the specimen would tend to be flexed while binding the membrane to the unsupported cap. Alternatively, the specimen can be set upon an inverted cap clamped to a ringstand and the membrane placed over the specimen and bound to the cap; then the specimen and cap can be inverted onto the base and the lower end of the membrane secured.

(3) Specimen dimensions not measured precisely. Dial gages or micrometers are helpful in obtaining precise measurements. When the specimen diameter is measured after being enclosed by the membrane, twice the thickness of the membrane must be subtracted from the measurement. The cross-section area of large specimens may be determined most satisfactorily from circumference measurements.

c. Q Test. (1) Changes in specimen dimensions upon application of chamber pressure. Partially saturated specimens will compress under the chamber pressure so the change in height, ΔH_o, due to the application of chamber height should be recorded. When this change in height is significant, the area of specimen before shear, A_c, should be computed according to the formula:

$$A_c = A_o \frac{H_o - 2\Delta H_o}{H_o}$$

as given in paragraph 6c(2).

(2) Rate of strain too fast.

(3) Water content determination after test not representative. Friction between the soil and the cap and base restrains the radial deformation at the ends of the specimen and this end restraint induces a non-uniform pore pressure distribution which, in turn, causes pore water migration within the specimen. For relatively impervious soils, a

significant migration of pore water could occur only in a test of long duration (such as S and some R tests); however, for more pervious soils, an appreciable redistribution of water content can occur within the short duration of a Q test. Therefore, it may be desirable to determine the water content of the end sections (about 1/6 of the height at each end) separately from the middle portion. Correlations of strength with water content should be based on the water content of the middle portion, though the dry weight of the entire specimen is needed to compute the initial soil properties.1

d. R Test. (1) Back pressure increments too large in relation to effective consolidation pressure.

(2) Back pressure increments applied too rapidly.

(3) Chamber and back pressures not precisely maintained during consolidation phase. Variations in either or both of these pressures (often much larger than the difference between them) can result in overconsolidation of the specimen.

(4) Specimen not completely consolidated before shearing.

(5) Rate of strain too fast.

(6) Excessive variations in temperature during shear. An increase in temperature will decrease the effective stress in a specimen during undrained shear. This danger, obviously, increases with the duration of the test.

(7) Specimen absorbed water from porous inserts at end of test. As in a consolidation test or a direct shear test, the specimen will absorb water from the porous inserts and drainage lines at the end of the R or S test no matter how rapidly the apparatus is disassembled and the specimen removed. To obtain an accurate water content determination at

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† A. Casagrande and S. J. Poulos, Fourth Report on Investigation of Stress-Deformation and Strength Characteristics of Compacted Clays, Soil Mechanics Series No. 74, Harvard University (Cambridge, Mass., October 1964).

the end of the test, the specimen should be allowed to swell completely under a small (2 or 3 psi) chamber pressure and the increase in volume measured by means of the burette. This volume change can then be used to correct the water content measured after the test.

e. S Test. (1) Rate of strain or rate of loading too fast.

(2) Inaccurate volume change measurements during shear. Where volume changes are measured using a burette, inaccuracies may result from incomplete saturation of the specimen, leakage, evaporation, or temperature fluctuations.

TRIAXIAL COMPRESSION TEST
(SPECIMEN DATA)

Date _____

Project _____

Boring No. _____ Sample No. _____

Type of Test				Confining Pressure			tons/sq ft
Test No.				Classification			
		Before Test				After Test	
		Specimen		Trimmings		Specimen	
Tare No.							
Weight, g: Tare plus wet soil							
Tare plus dry soil							
Water	W_w			W_{wo}		W_{wf}	
Tare							
Wet soil	W						
Dry soil	W_s						
Water content	w	%		w_o	%	w_f	%

Initial Condition of Specimen

Diameter, cm	D_o	Top		Center		Bottom	Avg
Height, cm	H_o		Volume of solids, cc	V_s			
Area, sq cm = $0.7854\ D_o^2$	A_o		Void ratio = $(V_o - V_s) \div V_s$	e_o			
Volume, cc = H_oA_o	V_o		Saturation, %	S_o			
Specific gravity of solids	G_s		Dry density, lb/cu ft	γ_d			

Condition of Specimen After Consolidation (R and S Tests)

Change in height during consolidation, in.	ΔH_o		Volume, cc = A_cH_c	V_c	
Height, cm = $H_o - 2.54\Delta H_o$	H_c		Void ratio = $(V_c - V_s) \div V_s$	e_c	
Area, sq cm	A_c		Saturation, %	S_c	

Condition of Specimen After Test (R and S Tests)

Diameter, cm	D_f	Top		Center		Bottom	Avg
Change in height during shear test, in.	ΔH		Volume, cc = A_fH_f	V_f			
Height, cm = $H_c - 2.54\Delta H$	H_f		Void ratio = $(V_f - V_s) \div V_s$	e_f			
Area, sq cm	A_f		Saturation, %	S_f			

$$W_s = \frac{W}{1 + \frac{w_o}{100}}, \quad V_s = \frac{W_s}{G_s}, \quad S_o = \frac{\frac{w_o}{100} \times \frac{W_s}{\gamma_w}}{V_o - V_s} \times 100, \quad S_c = \frac{\frac{w_c}{100} \times \frac{W_s}{\gamma_w}}{V_c - V_s} \times 100,$$

$$S_f = \frac{\frac{w_f}{100} \times \frac{W_s}{\gamma_w}}{V_f - V_s} \times 100, \quad \gamma_d = \frac{W_s}{V_o} \times 62.4, \quad A_c = A_o \frac{H_o - 2\Delta H_o}{H_o}$$

Remarks _____

_____

Technician _____ Computed By _____ Checked by _____

PLATE X-1 ENG FORM 1 JUN 65 3852

TRIAXIAL COMPRESSION (Q AND R) TEST
AXIAL LOADING DATA

Date _____

Project _____

Boring No. _____ Sample No. _____ Test No. _____

Type Test _____ Confining Pressure _____ tons/sq ft

Time	Elapsed Time min	Dial Reading 10^{-} in.	Cumulative Change (ΔH) 10^{-} in.	P Axial Load lb	Axial Strain* $\epsilon = \frac{\Delta H}{H}$	$1 - \epsilon$	$A_{corr} = \frac{A^{**}}{1-\epsilon}$ sq cm	Deviator Stress = $\frac{P}{A_{corr}} \times 0.465$ tons/sq ft

* Use H_o for Q tests and H_c for R tests.
** Use A_o for Q tests and A_c for R tests.

H_o _____ cm = _____ in. A_o _____ sq cm
H_c _____ cm = _____ in. A_c _____ sq cm

Test time to failure _____ min Type failure _____

Technician _____

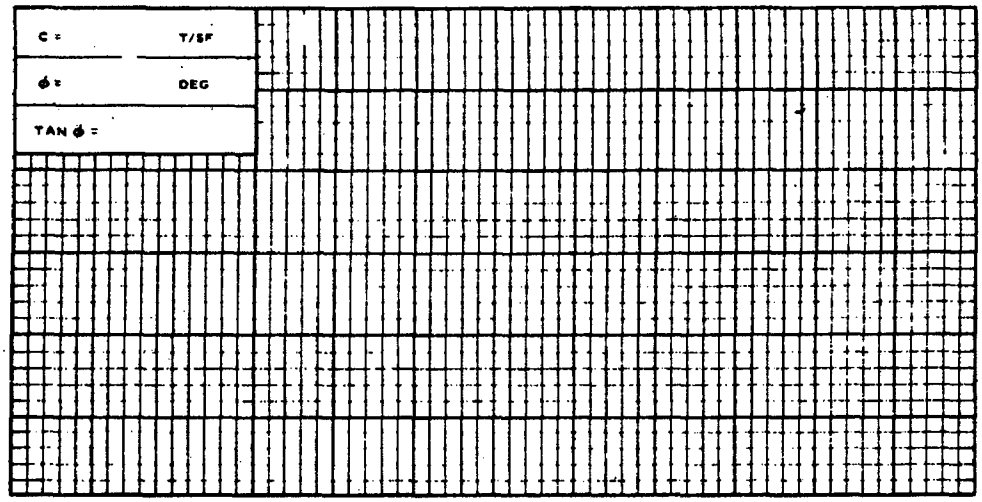
ENG FORM 3853
1 JUN 65

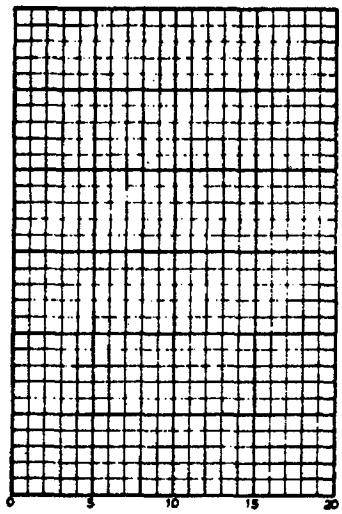
PLATE X-2

C = T/SF

ϕ = DEG

TAN ϕ =





SPECIMEN NO.						
INITIAL	WATER CONTENT, %	w_o				
	DRY DENSITY LB/ CU FT	γ_{d_o}				
	SATURATION, %	S_o				
	VOID RATIO	e_o				
BEFORE SHEAR	WATER CONTENT, %	w_c				
	DRY DENSITY LB/CU FT	γ_{d_c}				
	SATURATION, %	S_c				
	VOID RATIO	e_c				
	FINAL BACK PRESSURE, T/SQ FT	u_b				
MINOR PRINCIPAL STRESS, T/SQ FT		σ_3				
MAXIMUM DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{MAX}$				
TIME TO $(\sigma_1 - \sigma_3)_{MAX}$, MIN		t_f				
ULTIMATE DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{ULT}$				
INITIAL DIAMETER, IN.		D_o				
INITIAL HEIGHT, IN.		H_o				

CONTROLLED- TEST

DESCRIPTION OF SPECIMENS

LL	PL	PI	Gs	TYPE OF SPECIMEN	TYPE OF TEST

REMARKS:

PROJECT

BORING NO. SAMPLE NO.

DEPTH ELEV

LABORATORY DATE

TRIAXIAL COMPRESSION TEST REPORT

ENG FORM NO. 2089
REV JUNE 1970

PLATE X-3

TRIAXIAL COMPRESSION TEST
(BACK PRESSURE AND PORE PRESSURE DATA)
(Saturation)

Date _____

Project _____ Boring No. _____ Sample No. _____

Test No. _____ Consolidation Pressure _____ tons/sq ft = _____ psi

Increment No.	Time	Elapsed Time min	Chamber Pressure psi	Applied Back Pressure psi	Measured Pore Pressure psi	Burette Reading cc	Volume Change (ΔV) cc

Technician _____

ENG FORM 3854
1 JUN 68

PLATE X-4

TRIAXIAL COMPRESSION (R AND S) TEST
PRELIMINARY CONSOLIDATION

Date _____

Project _____

Boring No. _____ Sample No. _____ Test No. _____

Test Type _____ Consolidation Pressure _____ tons/sq ft

Date and Time	Elapsed Time min	Burette Reading cc	Volume Change (ΔV) cc	Dial Indicator Reading 10^{-} in.	Date and Time	Elapsed Time min	Burette Reading cc	Volume Change (ΔV) cc	Dial Indicator Reading 10^{-} in.

Dial indicator reading before consolidation _____ in.
Dial indicator reading after consolidation _____ in.
Change in height during consolidation (ΔH_o) _____ in. = _____ cm

Remarks _____

Technician _____

PLATE X-5 ENG FORM 1 JUN 65 3855

TRIAXIAL COMPRESSION (S) TEST
AXIAL LOADING DATA

Date _____

Project _____

Boring No. _____ Sample No. _____ Test No. _____

Specimen No. _____ Consolidation Pressure _____ tons/sq ft

Date and Time	Dial Reading 10^{-4} in.	Cumulative Change (ΔH) 10^{-4} in.	Burette Reading cc	Volume Change (ΔV) cc	P Axial Load lb	Axial Strain $\epsilon = \frac{\Delta H}{H_c}$	$C\epsilon$	$1 - C\epsilon$	$A'_{corr} = \frac{A_c}{1 - C\epsilon}$ sq cm	Stress $\frac{P}{A'_{corr}} \times 0.465$ tons/sq ft

Area of specimen after test, A_f = _____ sq cm Axial strain at end of test, ϵ_e = _____

Area of specimen at end of test, $A_e = \frac{A_c}{1 - \epsilon_e}$ = _____ sq cm $C = \frac{A_f}{A_e}$ = _____

Test time to failure, t_f = _____

Remarks _____

Technician _____

ENG FORM 3856
1 JUN 65

PLATE X-6

APPENDIX XA:

CYCLIC TRIAXIAL TESTS

1. PRINCIPLES OF CYCLIC TRIAXIAL TESTING. The stress-controlled cyclic triaxial test is used to evaluate the liquefaction potential and strength of a soil under shear stresses representative of those induced by an earthquake. For horizontal soil deposits, the triaxial specimen is consolidated isotropically, and the cyclic shear stresses generated by an earthquake are simulated in the laboratory by cycling shear stresses along the 45-deg plane of a triaxial compression specimen under undrained conditions. For sloping ground surfaces, the triaxial specimen is consolidated anisotropically. Hence, the test is comparable to an R test with pore pressure measurements (see Appendix X, TRIAXIAL COMPRESSION TESTS) with the notable exception that the load is cyclically applied to the specimen such that the specimen is subjected to alternating cycles of vertical compression and extension about some ambient stress state, which produce corresponding cyclic shear stresses on the 45-deg plane, as illustrated in Figure 1.† Cyclic shear stresses in the test on anisotropically consolidated specimens are depicted in Figure 2.†† Thus, the major difference between cyclic triaxial and conventional triaxial equipment is a rigid fixed connection between the specimen cap and the loading piston.

The consequence of cyclic loading under undrained conditions is generally an increase in the pore water pressure, which causes the

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† M. L. Silver, "Laboratory Triaxial Testing Procedures to Determine the Cyclic Strength of Soils," Report No. NUREG-31, 1976, U. S. Nuclear Regulatory Commission, Washington, D. C.

†† M. L. Silver et al., "Cyclic Triaxial Strength of Standard Test Sand," Journal of Geotechnical Engineering, ASCE, Vol 102, GT5, May 1976.

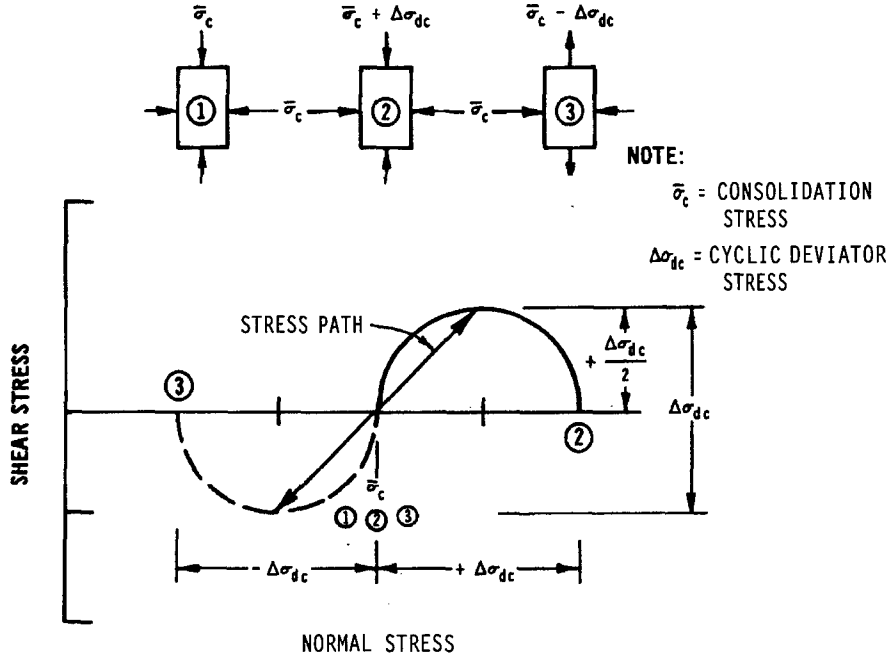


Figure 1. Mohr's circle of total stress representation for a cyclic triaxial strength test for an isotropically consolidated specimen (after Silver, 1976)

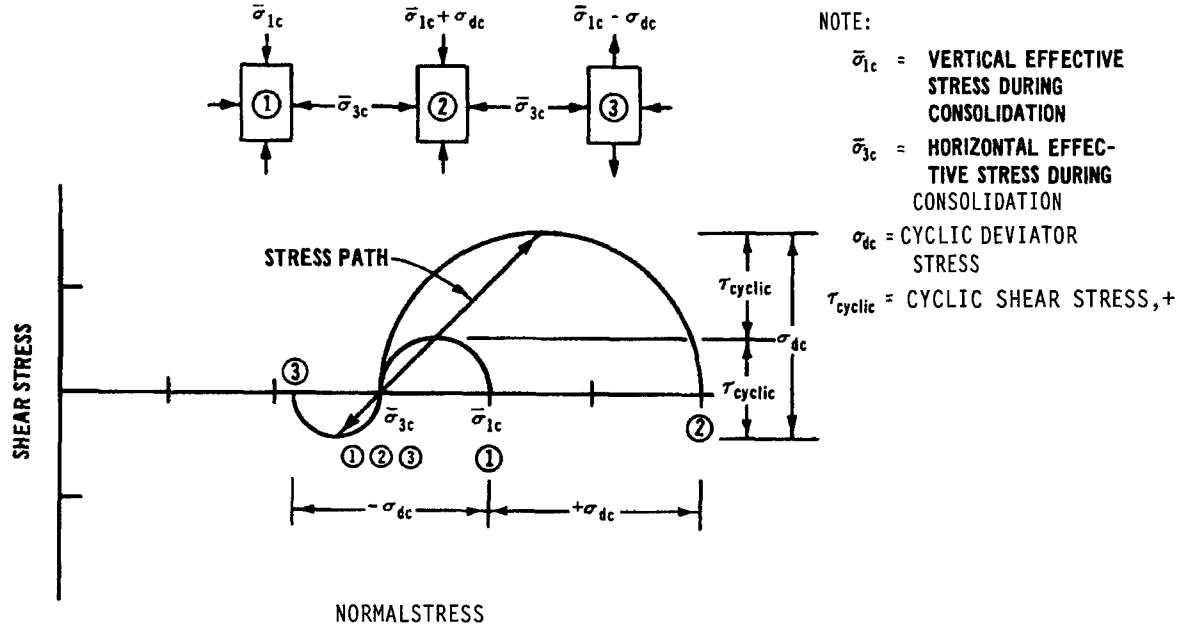


Figure 2. Mohr's circle of total stress representation for a cyclic triaxial test for an anisotropically consolidated specimen (after Silver et al., 1976)

effective stress to decrease and the cyclic deformations of the specimen to increase. In this test, initial liquefaction is defined to occur when the pore water pressure first equals the confining pressure, i.e., a condition where the effective stress is zero, with failure defined in terms of a limiting cyclic strain, generally either 5 or 10 percent from peak to peak. Peak-to-peak strain is usually referred to as double amplitude strain and is the total strain that the specimen undergoes on consecutive peaks of the strain-time trace. For isotropically consolidated specimens, these strains will be alternatively compressional and extensional; however, in the case of anisotropically consolidated specimens, these strains may be alternative compression-extension or permanent compression strain only.

The applied cyclic axial stress, $\sigma_1 - \sigma_3$ or P/A (see Fig. 1 of Appendix X, TRIAXIAL COMPRESSION TESTS), is termed the cyclic deviator stress, σ_{dc}, which is alternatively positive and negative about some ambient stress state. For isotropically consolidated specimens, test results are expressed in terms of the cyclic stress ratio, $\sigma_{dc}/2\sigma_{3c}$, which is the cyclic shear stress, $\sigma_{dc}/2$, normalized by the confining pressure σ_{3c}. For anisotropically consolidated specimens, test results may be expressed in terms $\tau_{cyclic}/\sigma_{fc}$, where τ_{cyclic} is the cyclic shear stress on the failure plane $(45^{\circ} + \phi'/2)$ and σ_{fc} is the normal stress on this plane during consolidation. The value of ϕ' can be estimated or determined from static tests. Research† has shown that the cyclic strength depends on density, confining

† J. P. Mulilis, R. C. Horz, and F. C. Townsend, "The Effect of Cyclic Triaxial Testing Techniques on the Liquefaction Behavior of Monterey No. 0 Sand," Miscellaneous Paper MP S-76-6, Apr 1976, U. S. Army Engineer Waterways Experiment Station, CE, Vicksburg, Miss.

F. C. Townsend, "A Review of Factors Affecting Cyclic Triaxial Tests," Dynamic Geotechnical Testing, ASTM STP 654, Jun 1977.

pressure, applied cyclic shear stress, stress history, specimen preparation procedure, and uniformity and shape of cyclic wave form; hence, close attention must be given to testing details and equipment.

2. SPECIMENS. In most cases, only high-quality undisturbed samples should be used for testing since sand fabric (particle orientation) has a considerable effect on cyclic strength. Presently, no techniques are available for duplication of the in situ fabric by laboratory reconstitution. If there is an insufficient number of undisturbed specimens available for a testing program, then correlations between undisturbed and reconstituted specimens are necessary to provide data over the entire range of conditions desired.

3. APPARATUS. All the equipment listed in Appendix X, TRIAXIAL COMPRESSION TESTS, for performing triaxial compression R tests with pore pressure measurements, is required for cyclic triaxial tests in addition to the following special equipment: (1) cyclic loading equipment, (2) electronic transducers and high-speed recorders for data acquisition, and (3) provisions for fixing the specimen cap to the loading piston for the extension portion of the loading cycle.

a. Loading Devices. Dynamic loading equipment used for stress-controlled cyclic triaxial tests must be capable of applying a uniform sinusoidal load at a frequency of 1 Hz or greater. The equipment may be either (1) a closed-loop electrohydraulic system or (2) a pneumatic system.† Both systems basically consist of a pressurized fluid (oil or air) whose pressure into a loading actuator is varied by a servovalve or regulator commanded by a control unit. The system should be capable of maintaining the cyclic deviator load constant throughout the test, i.e., must provide sufficient hydraulic oil or air to permit the loading piston to follow the sudden and rapid specimen deformation at

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†C. K. Chan and J. P. Mulilis, "A Pneumatic Sinusoidal Loading System," Journal of Geotechnical Engineering, ASCE, Vol 102, No. GT3, Mar 1976.

liquefaction. The loading device must maintain uniform cyclic peak loadings throughout the test. Unsymmetrical compression-extension load peaks, nonuniformity of pulse duration, "ringing," or overshoot must not exceed tolerances illustrated in Figure 3.† In these cases, differences in peak compressive and extension loads or durations greater than 10 percent are unacceptable. Both "ringing" and overshoot can induce abnormally rapid pore pressure rises affecting specimen failure.

A problem common to most cyclic loading equipment is the reduction in cyclic load at the onset of large specimen deformation. For the test to be meaningful, the load must be symmetrical in extension and compression up to peak-to-peak strains of 20 percent; and the peak-to-peak load should not decrease by more than 20 percent from the initially applied values until the specimen peak-to-peak strains exceed 10 percent. The equipment must also be able to apply the cyclic load about an initial static load on the loading piston. This static load counteracts the uplift pressure, which results, from the chamber pressure acting on the reduced area of the specimen cap because of the fixed loading piston connection. Further, in the case of anisotropically consolidated specimens, the static load applies some of the consolidation stress.

b. Specimen Cap. The specimen cap should be of a lightweight noncorrosive material equipped with porous metal or porous stone inserts and drainage connections. The cap can be similar to those in paragraph 3c of Appendix X, TRIAXIAL COMPRESSION TESTS, except (1) provisions for fixing the loading piston rigidly to the cap must be provided and (2) specimen diameter should be 2.8 in. or larger. The most common connection is simply straight threads backed by a shoulder on the piston, which tightens up against the loading piston.

c. Triaxial Compression Chamber. The triaxial compression

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† Silver, op. cit.

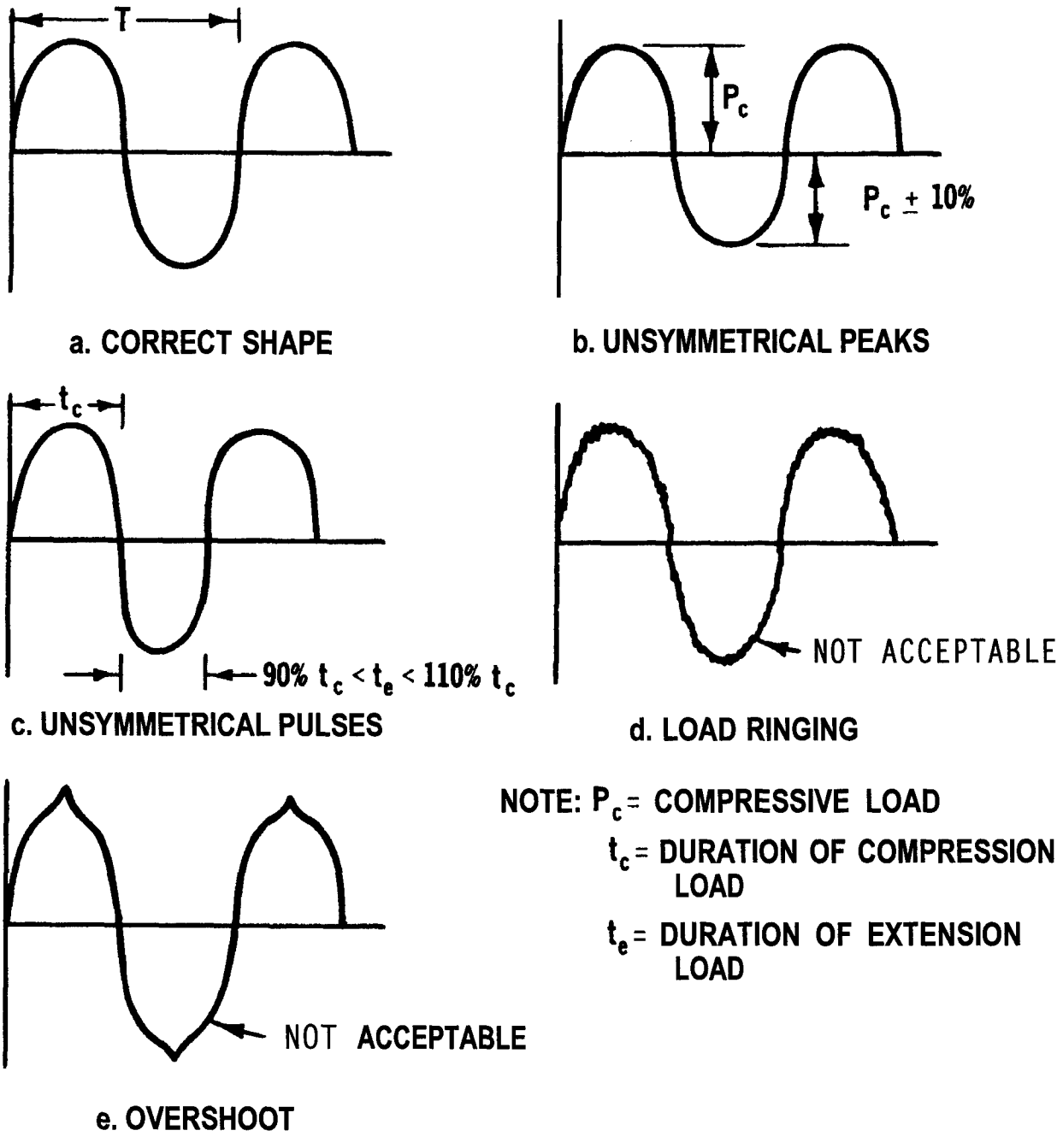


Figure 3. Examples of acceptable and unacceptable loading wave forms for cyclic triaxial strength tests (after Silver, 1976)

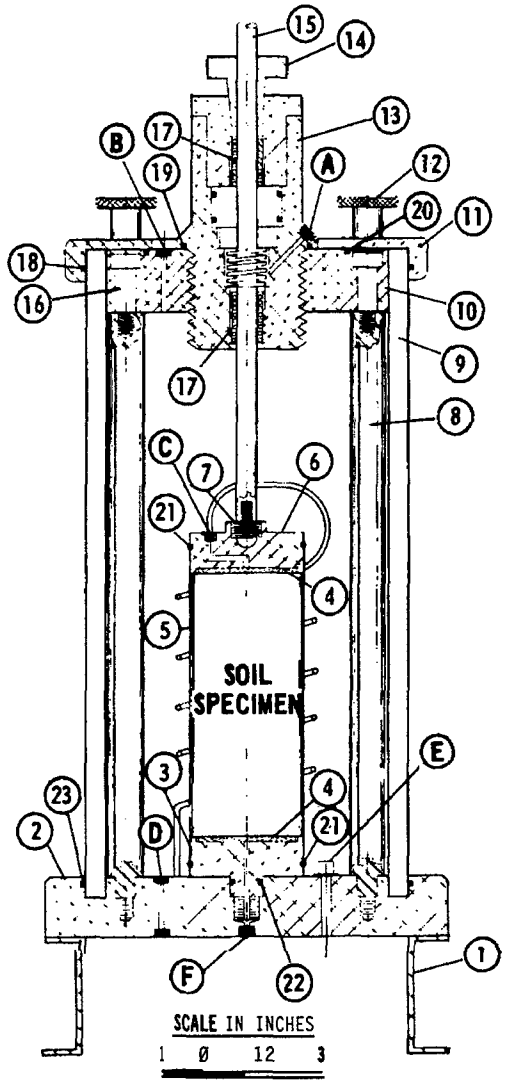
chamber consists of a headplate and a baseplate separated by a transparent plastic pressure cylinder. A recent design of a triaxial compression chamber used for cyclic triaxial testing 2.8-in.-diameter specimens is shown in Figure 4. Essential features of this preferred design, compared with that illustrated in paragraph 3b of Appendix X, TRIAXIAL COMPRESSION TESTS, are (1) internal tie-rods and external plastic chamber and (2) a hydrostatic seal with linear ball bushings. The internal tie-rods and external plastic chamber facilitate specimen cap alignment, loading piston to loading machine alignment, and connection of the loading piston to the specimen. Several designs of hydrostatic seal have proven effective.† All incorporate a close tolerance floating sleeve, which allows air to leak between the sleeve and loading piston, eliminating metal-to-metal contact between loading piston and cell and minimizing the speed and pressure-dependent friction inherent in O-rings and bushings.

The maximum acceptable piston friction without applying load corrections is ±2 percent of the maximum cyclic load. Piston friction values can be evaluated by calibration performed by (1) assembling the entire triaxial chamber without a specimen, (2) connecting the chamber to the loading equipment, and (3) recording the load cell output for the complete range of loading piston displacements for the chamber pressures that will be used.

d. Loading Piston to Loading Equipment Connection. The ideal loading piston to loading equipment connector is one that (1) is easy to install; (2) does not slip under cyclic load or vibration; (3) does not twist the specimen during connection; and (4) eliminates the effects of any eccentricity between the line of action of the loading equipment and loading piston. Figure 5 presents three examples of successful connectors: the union nut, split nut, and spherical clamp.

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† C. K. Chan, "Low Friction Seal System," Journal of Geotechnical Engineering Division, ASCE, Vol 101, GT9, Sep 1975.

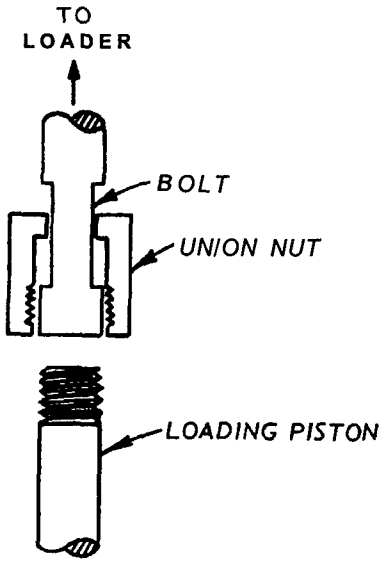


TRIAXIAL COMPRESSION CHAMBER

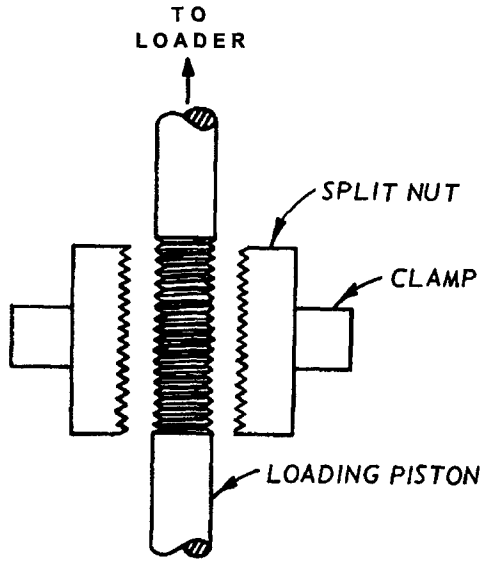
A. HYDROSTATIC SEAL AIR SUPPLY PORT
B. VENT
C. TOP DRAINAGE PORT
D. CHAMBER FLUID DRAIN
E. TRANSDUCER CABLE CONNECTOR
F. BOTTOM DRAINAGE PORT

PART NO.	NAME	MATERIAL	QUANTITY
1	CHAMBER FOOT	ALUMINUM CHANNEL	2
2	CHAMBER BASE	ALUMINUM PLATE	1
3	SPECIMEN BASE	ALUMINUM	1
4	POROUS PLATE	STAINLESS STEEL	2
5	SPECIMEN MEMBRANE	RUBBER	1
6	SPECIMEN CAP	ALUMINUM	1
7	SPECIMEN CAP CONNECTOR	BRASS	1
8	BEARING SUPPORT ROD	STAINLESS STEEL	2
9	CYLINDER	CLEAR ACRYLIC	1
10	CHAMBER CAP	ALUMINUM	1
11	CYLINDER SEALING RING	ALUMINUM	1
12	LOCKING KNOBS	STEEL	3
13	AIR BEARING ASSEMBLY	ALUMINUM	1
14	PISTON LOCK	BRASS	1
15	LOADING PISTON	STAINLESS STEEL	1
16	SHOULDER SCREW	STEEL	2
17	BALL BUSHING	STAINLESS STEEL	2
18	O-RING	RUBBER	1
19	O-RING	RUBBER	1
20	O-RING	RUBBER	1
21	O-RING	RUBBER	2
22	O-Ring	RUBBER	1
23	O-RING	RUBBER	1

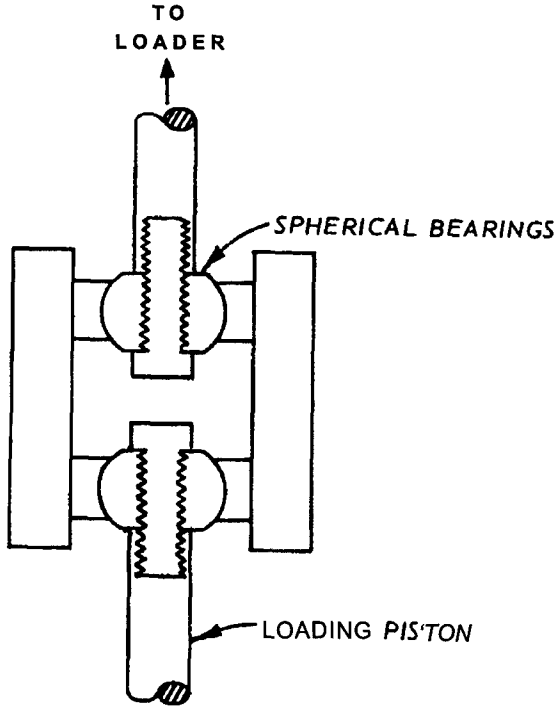
Figure 4. Triaxial compression chamber used for cyclic triaxial tests (working drawings for design shown available from Soil Mechanics Division, Geotechnical Laboratory, U. S. Army Engineer Waterways Experiment Station)



a. UNION NUT



b. SPLIT NUT



c. SPHERICAL CLAMP

Figure 5. Examples of successful connections between triaxial cell loading piston and cyclic loading actuator (after Silver, 1976)

e. Recording Equipment. Specimen behavior in stress-controlled cyclic triaxial tests is evaluated from continuous time records of applied load, specimen deformation, and change in pore water pressure. Commonly, these parameters are recorded on a multichannel strip-chart recorder, as illustrated in Figure 6. Analog to digital data acquisition systems may be used provided that data can be converted later into a convenient form for data analysis and interpretation. Fast recording system response is essential if accurate specimen performance is to be monitored when failure conditions are approached. It is recommended that the response characteristics in Table 1 be satisfied.

For analog strip-chart recording equipment, the magnitude of the load, deformation, and pore water pressure (chamber pressure recording is optional) recorder trace must be of sufficient amplitude and time resolution to enable accurate data reduction. Resolution of each variable should be better than 2 percent of the maximum value being measured. To take advantage of recorder accuracy and for subsequent data analysis, a recorder speed of 2-4 cycles per in. of recording paper is acceptable. The clarity of the trace with respect to the background should provide sufficient contrast and minimum trace width, so that the minimum resolution of 2 percent of the maximum value of the recorded parameter is maintained, and the trace may be included in reports.

f. Measurement Transducers. Load, displacement, and pore water pressure transducers are required to monitor specimen behavior during stress-controlled cyclic triaxial tests; provisions for monitoring the chamber pressure are optional. Each of these transducers must have the required capacity to ensure that the full range of specimen characteristics are monitored; at the same time, they must have the required sensitivity to ensure that small changes in specimen behavior are properly measured and recorded.

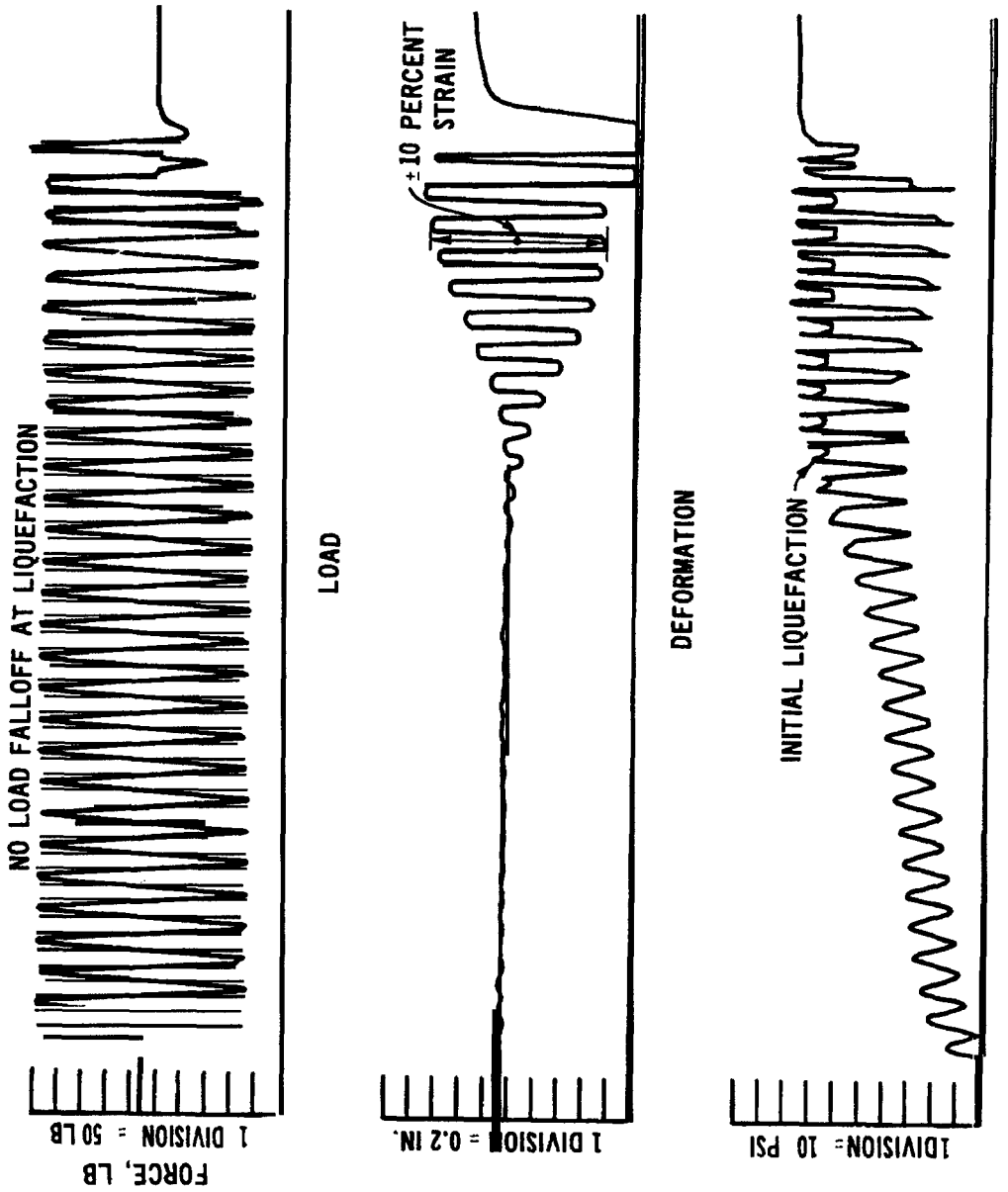


Figure 6. Analog recordings of load, deformation, and pore pressure during cyclic triaxial test

Table 1

Data Acquisition

Minimum Response Characteristics for
Cyclic Triaxial Strength Tests

1. Analog Recorders

Recording Speeds: 0.5 to 50 cm/sec (0.2 to 20 in./sec)

System Accuracy (include linearity and hysteresis): 0.5%(1)

Frequency Response: 100 Hz

2. Measurement Transducers

	Load Cell	Displacement Transducer (LVDT)(2)	Pore Pressure
Minimum sensitivity, mv/v	2	0.2 mv/0.025 mm/v (AC LVDT) 5 mv/0.025 mm/v (DC LVDT)	2
Nonlinearity, % full scale	± 0.25	± 0.25	± 0.5
Hysteresis, % full scale	± 0.25	0.0	± 0.5
Repeatability, % full scale	± 0.10	± 0.04	± 0.5
Thermal effects on zero shift or sensitivity, % of full scale/°C(°F)	± 0.005 (± 0.025)	--	± 0.02 (± 0.01)
Maximum deflection at full rated value in mm (in.)	0.425 (0.005)		
Volume change characteristics (cu in./psi)			1.0×10^{-4}

Note: (1) System frequency response, sensitivity, and linearity are functions of the electronic system interfacing, the performance of the signal conditioning system used, and other factors. It is therefore a necessity to check and calibrate the above parameters as a total system and not on a component basis.

(2) LVDT's, unlike strain gauges, cannot be supplied with meaningful calibration data. System sensitivity is a function of excitation frequency, cable loading, amplifier phase characteristics, and other factors. It is necessary to calibrate each LVDT-cable-instrument system after installation, using a known input standard.

(1) Load transducers. · The total load capacity of the load transducer (load cell) should be of the proper order of magnitude with respect to the maximum total loads to be applied to the test specimen. Generally its capacity should be no greater than five times the total maximum load applied to the test specimen to ensure that the necessary measurement accuracy is achieved. The minimum performance characteristics of the load cell are presented in Table 1.

A rigid load cell is required to avoid resonance problems that can develop with closed-loop electrohydraulic loading systems.

Miniature lightweight load cells are used to prevent inertia effects during cycling. The load cell is normally placed outside the triaxial chamber. If the load cell is located inside the triaxial chamber, special provisions must be provided to either pressurize the load cell or normalize the effects of cell pressure to ensure that load readings are not affected by the cell pressure. In all cases, the response and performance of a load cell located inside the pressure chamber must be documented.

(2) Deformation measurement. The stress-controlled cyclic triaxial test requires a deformation transducer with a high resolution and small range during the initial portion of the test and less resolution but large range during the final portion of the test. The linear variable differential transformer (LVDT) is generally considered to be the most suitable deformation transducer for the test, and its minimum specifications will be discussed. However, other displacement measuring devices, such as eddy current sensors or optical methods, may be used if they meet the required performance criteria. Potentiometer type deformation transducers are not recommended because they are easily damaged if their maximum travel is exceeded.

The displacement transducer must have a range of at least 20-30 percent of specimen height but should not exceed 60 percent of specimen height. The specifications for this transducer, representing

levels on nonlinearity, hysteresis, and repeatability, are also presented in Table 1.

Accurate deformation measurements in dynamic triaxial tests require that the LVDT be properly mounted to avoid excessive mechanical system compression between the load frame, the triaxial cell, the load cell, and the loading piston. Thus, it is recommended that the LVDT be located either on the actuator rod or on the loading piston attached to the test specimen.

(3) Pore water pressure measurements. Pore water pressures may be individually measured in the drainage line(s) leading to either (or both) the specimen cap or base. However, more reliable measurements may be achieved if pore water pressure is measured at both the cap and base simultaneously by having a connection in the drainage lines between the specimen and the pressure transducer. The use of a differential pressure transducer will facilitate data reduction.

A rigid pore water pressure measuring system is required. To achieve reliable measurements, it is recommended that pore water pressure transducer volume change not exceed 1×10^{-4} cu in. per psi and that the transducers have the sensitivity and performance characteristics summarized in Table 1. Moreover, the rigidity of all the assembled components of the pore water pressure measurement system should not exceed 1×10^{-4} cu in. per psi. In general, stiff, small-diameter tubing of metal or Saran, short-tubing runs, and high-quality ball valves are required to meet this specification. In all cases, the rigidity of the entire pore water pressure system, including the transducer, must be ascertained and documented. Methods for measuring the rigidity of the entire pore water pressure system are presented

by Wissa (1969)† and Bishop and Henkel (1962).††

g. Saturation Equipment. The apparatus for back-pressure saturation described in Appendix X is adequate provided the optional vacuum source with a vacuum regulator is included in the setup. The vacuum source must have a means of regulation for those instances where the effective consolidation stress to be applied to specimens is less than the pressure difference suppliable by the vacuum. Carbon dioxide gas (CO_2) has been found very helpful in obtaining complete saturation in those instances where a full vacuum cannot be applied. The CO_2 is permitted to seep up through the specimen prior to seepage saturation.

h. Tamping Rod for Moist Tamping Specimen Preparation. (Optional) The "standard" moist tamping specimen preparation procedure uses a tamping foot, whose diameter is one-half that of the specimen. Figure 7 shows a sketch of a tamping foot for a 2.8-in.-diameter specimen. The tamping rod consists of a 3/4-in.-diameter steel rod 12 in. long with a 1.4-in.-diameter steel tamping foot attached to the end.

4. TESTING PROCEDURES. Specimen preparation, back-pressure saturation, and consolidation procedures specified in paragraphs 4b and 7b(1) through 7b(7), Appendix X, TRIAXIAL COMPRESSION TESTS, are adequate and pertinent. However, preferred techniques for handling undisturbed samples of cohesionless soils, preparing remolded specimens for a standard test calibration procedure,* and anisotropic consolidation are as follows:

a. Reconstituted Specimen Preparation (Moist Tamping). The procedure is as follows:

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† A. E. Z. Wissa, "Pore Pressure Measurement in Saturated Stiff Soils," Journal of Soil Mechanics and Foundations Engineering, ASCE, Vol 95, No. SM4, July 1969.

†† A. W. Bishop and D. J. Henkel, The Measurement of Soil Properties in the Triaxial Test, 2nd ed., London, Edward Arnold Ltd., 1962.

* Mulilis et al., op. cit.; and Silver, op. cit.

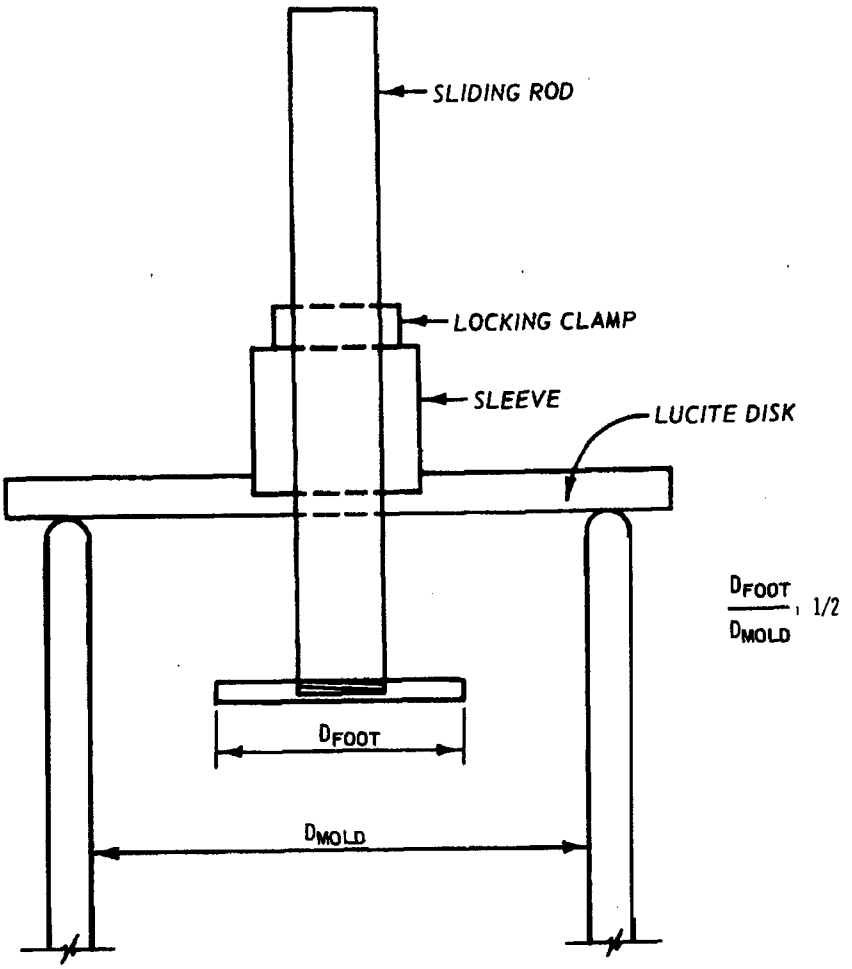


Figure 7. Tamping rod

(1) Adjust the water content of the material, so, that the initial degree of saturation of the compacted material will be between 20 and 70 percent.

(2) Place the forming jacket, with membrane inside, over the specimen base of the triaxial compression chamber.

(3) Evacuate the air between the membrane and inside face of the forming jacket, so that the membrane is taut and flush against the face.

(4) Determine the inside diameter and height of the mold to within 0.001 in. for 2.8-in.-diameter specimens, and calculate the

volume based on these measurements.

(5) Select the number of layers to be used in the preparation of the specimen. The maximum thickness should not exceed 1.0 in. for specimens having diameters less than 4.0 in.

(6) Starting with the bottom layer, compact each layer to the desired density using the tamping rod. The locking collar on the rod must be adjusted to produce the desired height to within 0.001 in. for each layer. This is 'most easily done by adjusting the travel of the foot for each layer using a set of gage blocks.

(7) A procedure of variable compaction, such as that described in paragraph 4b(4), Appendix X, TRIAXIAL COMPRESSION TESTS, may be used to construct uniform specimens. An alternative and similar procedure consists of building the specimen in equal thickness layers but placing each successive layer in a slightly denser state. For example, an increase of one percentage point of relative density per layer (the top layer placed to the desired average relative density for the entire specimen) has been found satisfactory for medium dense sand.

(8) After the forming jacket is filled to the desired height, place the specimen cap on top of the specimen, pull the membrane end over the cap, and fasten with O-rings. Carefully apply a vacuum to the interior of the specimen while observing the response of the specimen pressure transducer. Adjust the vacuum regulator as necessary to assure that the differential pressure supplied by the vacuum is not greater than the intended effective consolidation stress. A vacuum gage located between the vacuum regulator and valve K (see Fig. 16 of Appendix X) simplifies this procedure by indicating the differential supplied by the vacuum while valve K is still closed. With a vacuum on the interior of the specimen, remove the forming jacket and measure the specimen dimensions.

b. Undisturbed Specimen. Preparation. Sampling, transporting,

and testing undisturbed specimens of loose free-draining cohesionless materials so as to minimize disturbance is an extremely difficult task. However, the current practice is as follows:

(1) Obtain samples using a fixed-piston sampler and drilling mud.

(2) Place perforated packers at both ends of the sampling tube and allow the tube to drain in a vertical position. Drainage of the tube should be monitored and, if insufficient, can be aided by the application of a low vacuum, i.e. 5 in. (2.5 psi) of mercury (Hg), to the bottom packer.

(3) After drainage, the sample may be carefully transported to the laboratory, or alternatively frozen in the field using dry ice or liquid nitrogen and transported to the laboratory in a frozen condition. Free-draining samples, which contain no silt or clay layers that would form ice lenses upon freezing, should be transported frozen. It is important to monitor the sample length at all stages (after sampling, after drainage, after freezing, after transportation, and again before testing). Samples should be stored vertically, and storage time at the laboratory minimized to prevent corrosion of the tubes.

(4) (Optional) Keeping the sample frozen, X-ray the tube to observe possible layering of the sample, voids, gravels, etc., in the tube† and to facilitate selection of 7-in. sections for testing.

(5) If a milling machine is available, the tube may be cut lengthwise at two diametrically opposite places, using a rapid feed, and then cut into sections with an electric hacksaw. If a milling machine is not used, the desired section is cut with an electric hacksaw or a tube cutter with stiffening collars. The cut ends of the tube are then cleaned

† W. F. Marcuson III and E. L. Krinitzky, "Dynamic Analysis of Ft. Peck Dam," Technical Report S-76-1, U. S. Army Engineer Waterways Experiment Station, Nov 1976, Vicksburg, Miss.

of burrs, and the specimen is pushed from the tube after having been permitted to thaw slightly.

(6) The ends of the specimen should be trimmed smooth and perpendicular to their length using a mitre box or other trimming device. The specimen is then placed in the triaxial chamber and enclosed in a rubber membrane; then, a small vacuum (always less than $\bar{\sigma}_3$) is applied to hold it firmly while thawing. It is usually necessary to place the sample in a freezer at times during the cutting and trimming process to ensure that the specimen remains frozen until enclosed in the membrane.

(7) Dimensions of the specimen are taken to calculate the initial volume. If frozen, measurements are checked after the specimen has thawed. There should be no significant volume change; otherwise, freezing or transportation have caused sample disturbance.

c. Specimen Measurement. Inasmuch as density greatly influences the cyclic triaxial strength, it is imperative that accurate density determinations and volume change measurements be made during saturation and consolidation. For this reason, diameter measurements using a circumferential tape† to the nearest 0.001 in., height measurements to the nearest 0.01 in. at four locations, and weights to the nearest 0.1 g are recommended for 2.8-in.-diameter specimens.

d. Saturation. Saturation generally includes a stage during which water is allowed to seep into the specimen while its interior is under vacuum, followed by back pressuring to drive the remaining air in the specimen into solution with the pore water. A procedure found to be effective is as follows:

(1) After assembling the triaxial chamber, apply a small chamber pressure, 1-3 psi, to the specimen, and increase the vacuum to the interior, noting that any vacuum on the interior of the specimen

† Commercially available from PI Tape, Box 398, Lemon Grove, Calif. 92045.

contributes to the effective stress and that the difference between the vacuum and chamber pressures must not exceed the effective consolidation stress.

(2) After allowing the specimen to remain under vacuum for a period of approximately 5-15 min, permit de-aired water to slowly seep up through the specimen from the bottom. The upward movement of water must be slow enough to minimize entrapment of air pockets. The use of de-aired water further promotes the removal of air by solution. Flushing of water through the specimen should continue until gas bubbles no longer come from the upper drainage line.

(3) After seepage saturation, reduce the vacuum and increase the chamber pressure simultaneously until the specimen is at atmospheric pressure. Back pressure the specimen in steps and evaluate the degree of saturation at appropriate intervals using Skempton's pore pressure parameter B . To do this, close the drainage lines from the specimen, increase the chamber pressure 5 or 10 psi, and adjust the axial load to match the increase in chamber pressure. Observe the pore pressure increase, Δu , and calculate B , using the equation:

$$B = \frac{\Delta u}{\Delta \sigma_3}$$

The specimen shall not be considered adequately saturated unless the B value is greater than 0.95.

In some cases, a satisfactory degree of saturation cannot be achieved by ordinary vacuum seepage and back-pressure saturation techniques. This may be due to a weak vacuum source or to the need to keep the vacuum low to prevent prestressing the specimen. In the latter case, it is possible to apply a sufficient vacuum to the chamber so that there is no prestressing of the specimen when a full, i.e. 14.7 psi, vacuum is applied to the interior of the specimen.

In either of the cases where the vacuum is limited, CO_2† can be allowed to slowly seep upward from the bottom of the specimen, while the specimen is being formed or after it has been confined in the triaxial chamber. The CO_2 will displace the air in the specimen and, being much more soluble in water than air, will enable subsequent saturation steps to be carried out successfully.

During saturation, the change in height of the specimen should be measured to the nearest 0.001 in. In addition, during saturation and consolidation, an axial load must be applied to the loading piston, which is screwed into the specimen cap, to compensate for the uplift force on the loading piston. This static load will be expressed as

$$P_s = \sigma_3 A_r - \text{weight of apparatus below the load cell}$$

where

P_s = static load applied to specimen
σ_3 = chamber pressure
A_r = area of the loading piston

e. Consolidation. Isotropic consolidation is defined as

$$K_c = \frac{\bar{\sigma}_{1c}}{\bar{\sigma}_{3c}} = 1$$

where

K_c = consolidation ratio
$\bar{\sigma}_{1c}$ = vertical effective consolidation stress
$\bar{\sigma}_{3c}$ = horizontal effective stress

To consolidate the specimen isotropically, maintain the applied back-pressure constant and increase the chamber pressure until the

† Mulilis et al., op. cit.

difference between the chamber pressure and the back pressure equals the desired consolidation pressure. An axial load to counterbalance uplift due to increasing the chamber pressure must be applied. This may require incrementally applying the consolidation pressure to provide sufficient time to apply and adjust the counterbalancing uplift load. Changes in specimen height during consolidation should be measured to the nearest 0.001 in., and the change in specimen volume 'to the nearest 0.01 cc. A plot of burette and/or height readings versus logarithm of elapsed time, as shown in Figure 5 of Appendix VIII, CONSOLIDATION TEST, is optional.

Anisotropic consolidation is defined as

$$K_c = \frac{\bar{\sigma}_{1c}}{\bar{\sigma}_{3c}} \neq 1$$

This consolidation condition may be achieved by incrementally increasing the chamber pressure, $\bar{\sigma}_{3c}$, and by increasing the axial load to the required $\bar{\sigma}_{1c}$, plus uplift counterbalancing value, until the final $\bar{\sigma}_{3c}$ value is achieved. As stated previously, changes in specimen height and volume should be measured during consolidation.

Following consolidation, the drainage lines should be closed and the pore water pressure observed for a period of time to verify that no leaks in the membrane or pore water pressure system have occurred. If the time for consolidation exceeds 8 hr, the B value should be redetermined prior to cyclic loading.

f. Cyclic Loading. If it does not already exist, a large air pocket should be formed at the top of the triaxial chamber by draining water from the chamber but leaving sufficient water to cover the top of the specimen. The air pocket is required, so that the large, rapid piston movements in and out of the chamber at the onset of failure do not

create chamber pressure fluctuations. Compressed air instead of water may be used as the confining fluid provided saturation and consolidation do not exceed 8 hr. The procedure is as follows:

(1) Record test number and specimen identification on recorder trace.

(2) Zero the recorder and transducer outputs, and record calibration steps and scale factors.

(3) Close valve F (D should have been already closed) as shown in Figure 16 of Appendix X, TRIAXIAL COMPRESSION TESTS.

(4) Record the consolidation pressure, $\bar{\sigma}_c$, and estimate the magnitude of cyclic load to be applied for the desired stress ratio, SR, with the equation:

$$P_c = 2 \times \bar{\sigma}_{3c} \times SR \times A_c$$

where

P_c = estimated cyclic load to be applied to the specimen, lb

$\bar{\sigma}_{3c}$ = consolidation pressure (chamber pressure-back pressure), psi

SR = desired stress ratio

A_c = area of specimen after consolidation, sq in.

(5) Initiate cyclic loading with the first half cycle in compression using a 1- to 2-Hz sinusoidal load form. During cyclic loading, the chamber pressure is maintained constant, and the axial load, axial deformation, and change in pore water pressure (recording of chamber pressure is optional) are recorded with time. The load is cycled until either (a) the cyclic double amplitude strain exceeds 20 percent: (b) 500-load cycles or the maximum number required in the program is surpassed; (c) "necking" of the specimen is observed; or (d) the load wave form deteriorates beyond acceptable values.

In cases of anisotropic consolidation, where

$$\frac{\sigma_{dc}}{2\bar{\sigma}_{3c}} < \frac{K_c - 1}{2}$$

there will be no reversal in the direction of shear stress application, and the net axial stress will always be the major principal stress. For these conditions, the pore pressure does not usually increase sufficiently to produce initial liquefaction, rather the specimen tends to deform progressively. Conversely, where

$$\frac{\sigma_{dc}}{2\bar{\sigma}_{3c}} > \frac{K_c - 1}{2}$$

there will be a stress reversal and conditions of initial liquefaction usually occur.

g. Specimen Removal. Following cyclic testing, the specimen should be carefully removed from the triaxial cell in order that no particles are lost, then dried and weighed for dry unit weight calculations.

5. COMPUTATIONS. a. From the initial specimen data recorded on Plate XA-1, compute the initial water content, initial void ratio, initial dry density, and if required, the initial relative density, using equations presented in Appendixes I, WATER CONTENT - GENERAL, and II, UNIT WEIGHTS, VOID RATIO, POROSITY, AND DEGREE OF SATURATION.

b. From saturation and consolidation data, compute the values of Skempton's pore pressure parameter B, and record any specimen dimension changes during saturation, the static piston loads required to counterbalance uplift pressures, and the magnitude of applied back pressure (Plate XA-1 (Cont'd)).

c. From specimen dimension and volume changes during saturation and consolidation, compute the height, area, and dry density of the specimen after consolidation, using the following formulas:

$$A_s = \frac{A_o}{H_o}(H_o - 2\Delta H_s)$$

$$V_s = A_s(H_o - \Delta H_s)$$

$$H_c = H_o - \Delta H_s - \Delta H_c$$

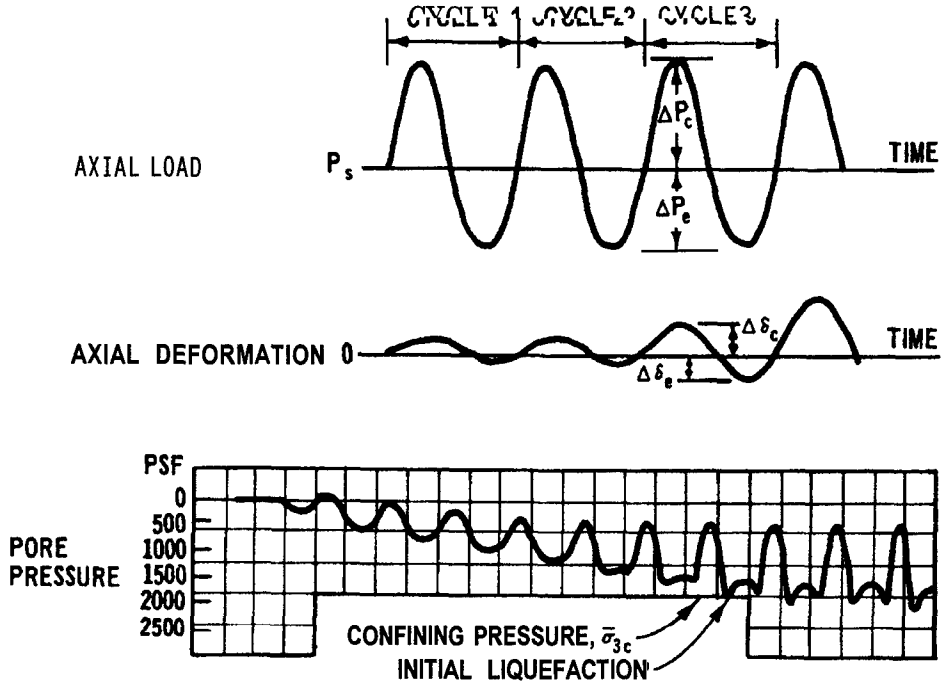
$$A_c = \frac{V_s - \Delta V_w}{H_c}$$

$$\gamma_{dc} = \frac{W_s}{A_c H_c} \times 62.4$$

where

A_s = area of specimen after saturation, sq cm
A_o = initial area of specimen, sq cm
H_o = initial height of specimen, cm
ΔH_s = change in height during saturation, cm
V_s = volume of specimen after saturation, cc
H_c = height of specimen after consolidation, cm
ΔH_c = change in height during consolidation, cm
A_c = area of specimen after consolidation, sq cm
ΔV_w = burette change during consolidation, cc
γ_{dc} = dry density after consolidation, pcf
W_s = weight of solids, g

d. After testing, use the data from the strip chart recorder and the relationships and definitions presented in Figure 8 to determine the cyclic stress, stress ratio, double amplitude strain (all based on consolidated specimen dimensions), and the induced pore water pressure; record these data on Plate XA-1. The number of cycles required to



DEFINITION OF CYCLIC STRESS, STRESS RATIO AND DOUBLE AMPLITUDE STRAIN

$$\text{CYCLIC STRESS, } \pm\sigma_{dc} = \frac{\Delta P_c \text{ t } \Delta P_e}{2A_c}$$

$$\text{CYCLIC STRESS RATIO, SR} = \frac{\pm\sigma_{dc}}{2\bar{\sigma}_{3c}}$$

$$\text{DOUBLE AMPLITUDE STRAIN, } \epsilon_{da}, \% = \frac{\Delta\delta_c + \Delta\delta_e}{H_c} \times 100$$

WHERE:

ΔP_c = PEAK CYCLIC LOAD IN COMPRESSION

ΔP_e = PEAK CYCLIC LOAD IN EXTENSION

$\pm\sigma_{dc}$ = CYCLIC DEVIATOR STRESS

$\pm\sigma_{dc}/2$ = CYCLIC SHEAR STRESS

δ_c = CYCLIC DEFORMATION IN COMPRESSION

δ_e = CYCLIC DEFORMATION IN EXTENSION

ϵ_{da} = DOUBLE AMPLITUDE AXIAL STRAIN, PERCENT

A_c, H_c = AREA AND HEIGHT AFTER CONSOLIDATION, RESPECTIVELY

Figure 8. Definitions and equations for computing cyclic stress, stress ratio, and double amplitude strain

achieve initial liquefaction (defined as 100 percent pore pressure response) and various strain amplitudes are also recorded in Plate XA-2.

The uniformity of the load trace into the failure state should be evaluated to ensure that the load uniformity criteria (see paragraph 3a) are achieved.

6. PRESENTATION OF RESULTS. a. Results of individual tests on specimens, which have been isotropically consolidated, are generally presented by plotting various percent (e.g. 5, 10, and 20) double amplitude strain versus number of cycles required to reach those strains. Since anisotropically consolidated specimens often reach failure strains in compression only, the various percentages of zero-to-peak compressive strain can be plotted versus number of cycles to reach those strains. In addition, pore pressure increase, $\Delta u / \bar{\sigma}_{3c}$, versus number of cycles to reach those percentages of increase (e.g. 25, 50, 75, and 100) can be plotted; the number of cycles to reach the given increase is denoted as N_{25i}, N_{50i}, N_{75i}, and N_i, respectively.

b. In analyzing the results of a series of isotropically consolidated cyclic triaxial strength tests, the number of cycles required to achieve initial liquefaction and various values of double amplitude strain are plotted (see Plate XA-2 and Fig. 9) versus stress ratio, SR, that is defined as

$$SR = \pm \frac{\sigma_{dc}}{2\,\bar{\sigma}_{3c}}$$

In the case of anisotropically consolidated cyclic triaxial strength tests, the definition of stress ratio is not appropriate, as $\bar{\sigma}_{3c}$ does not represent the total consolidation stress. Hence only $\pm\sigma_{dc}$ which is the cyclic deviator stress, versus log number of cycles for various values of strain (double amplitude and/or zero to peak compressional) are plotted on Plate XA-2.

For test series involving both anisotropically and isotropically

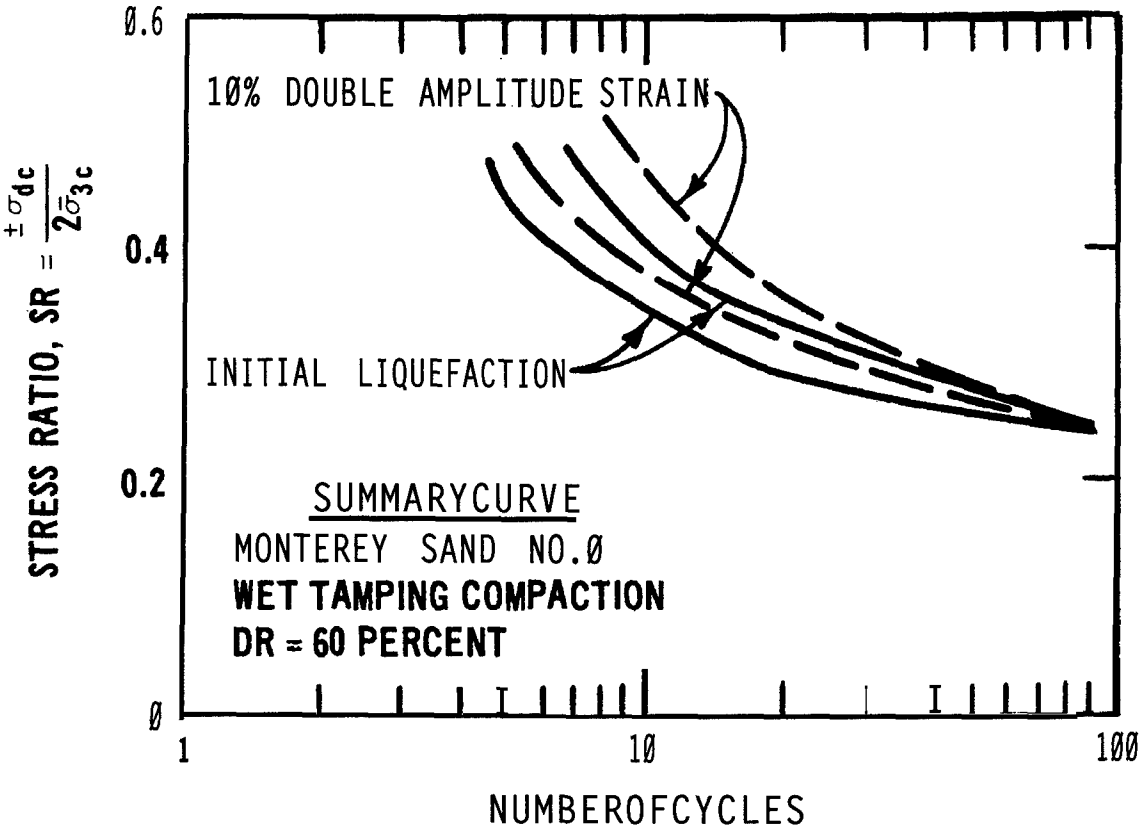
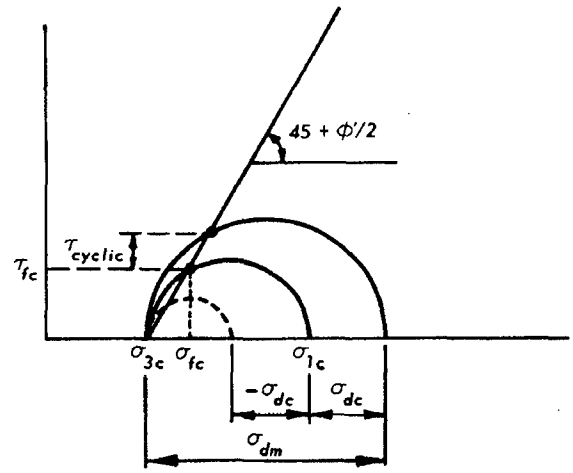


Figure 9. Range of cyclic triaxial strength values for initial liquefaction and 10 percent double amplitude strain

consolidated cyclic triaxial specimens, a summary figure of cyclic shear stress, τ_{cyclic}, on the failure plane ($45^{o} \pm \phi'/2$), required to produce a specified strain in a specified number of cycles versus the effective normal stress on the failure plane after consolidation, σ_{fc}, for various K_c ratios, as shown in Figure 10, is desirable. The ϕ' used in establishing the failure plane can be derived from static tests or may be assumed at some representative value.

c. To estimate the cyclic triaxial strength of undisturbed specimens, a density correction to account for differences between in situ densities and specimen density after consolidation can be made using



τ_{fc} = INITIAL SHEAR STRESS ON POTENTIAL FAILURE SURFACE

$$= \frac{\sigma_{3c}}{2} (K_c - 1) \cos \phi'$$

τ_{cyclic} = CYCLIC SHEAR STRESS DEVELOPED ON POTENTIAL FAILURE SURFACE

$$= \frac{\sigma_{dc}}{2} \cos \phi' \text{ OR } \left[\sigma_{dm} - \sigma_{3c} (K_c - 1)\right] \frac{\cos \phi}{2}$$

σ_{fc} = INITIAL NORMAL STRESS ON POTENTIAL FAILUE SURFACE

$$= \frac{\sigma_{3c}}{2} \left[(K_c + 1) - (K_c - 1) \sin \phi'\right]$$

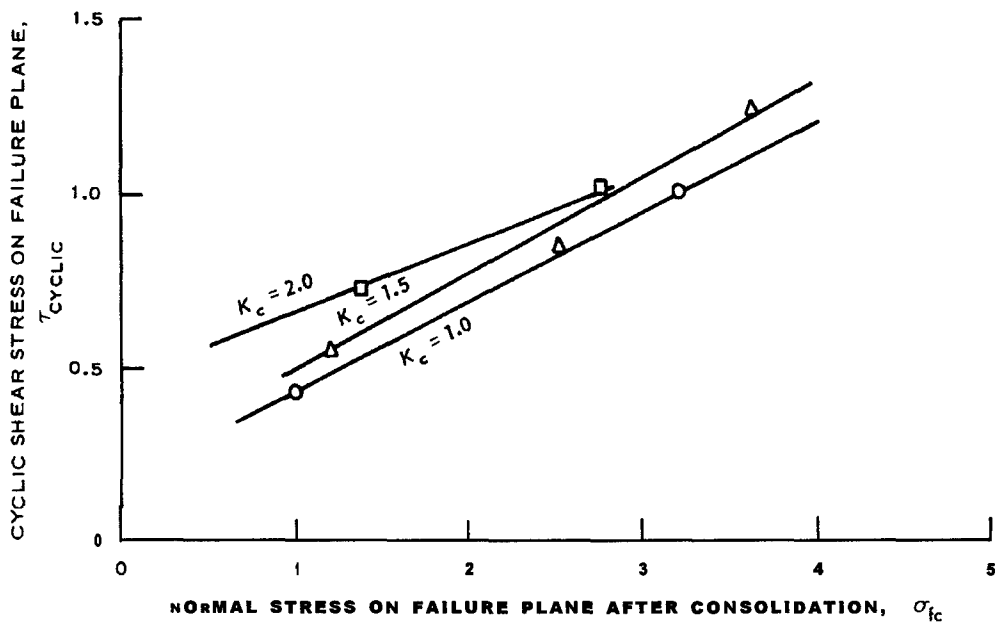


Figure IO. cyclic shear stress on failureplane, τ_{cyclic}, versus normal stress on failure plane after consolidation, σ_{fc}, for 5 percent strain in 5 cycles

the following empirical relationship,† provided D_d (laboratory) is near 50 percent:

$$\frac{\sigma_{dc}}{2\,\sigma'_o(D_d \text{ field})} = \frac{\sigma_{dc}}{2\,\sigma'_o(D_d \text{ lab})} \times \frac{D_d \text{ field}}{D_d \text{ lab}}$$

7. CALIBRATION OF EQUIPMENT AND TESTING PROCEDURES. Adherence to apparatus specifications and testing procedures are necessary but does not ensure that the cyclic triaxial test results from one laboratory will be comparable to those of other laboratories. Accordingly, a performance calibration has been developed using a standard test sand, Monterey No. 0,†† and standard test procedures.‡ It is recommended that each laboratory verify its performance by calibration using this standard test sand and techniques as follows:

a. Standard Test Sand - Monterey No. 0. Figure 11 presents the grain-size distribution and pertinent physical properties.

b. Specimen Preparation. Specimens, 2.8 in. in diameter by 6-7 in. high, are to be prepared to an initial density of 98.5 pcf (60 percent relative density) using the moist tamping procedure and moist tamping rod described in paragraphs 3h and 4a.

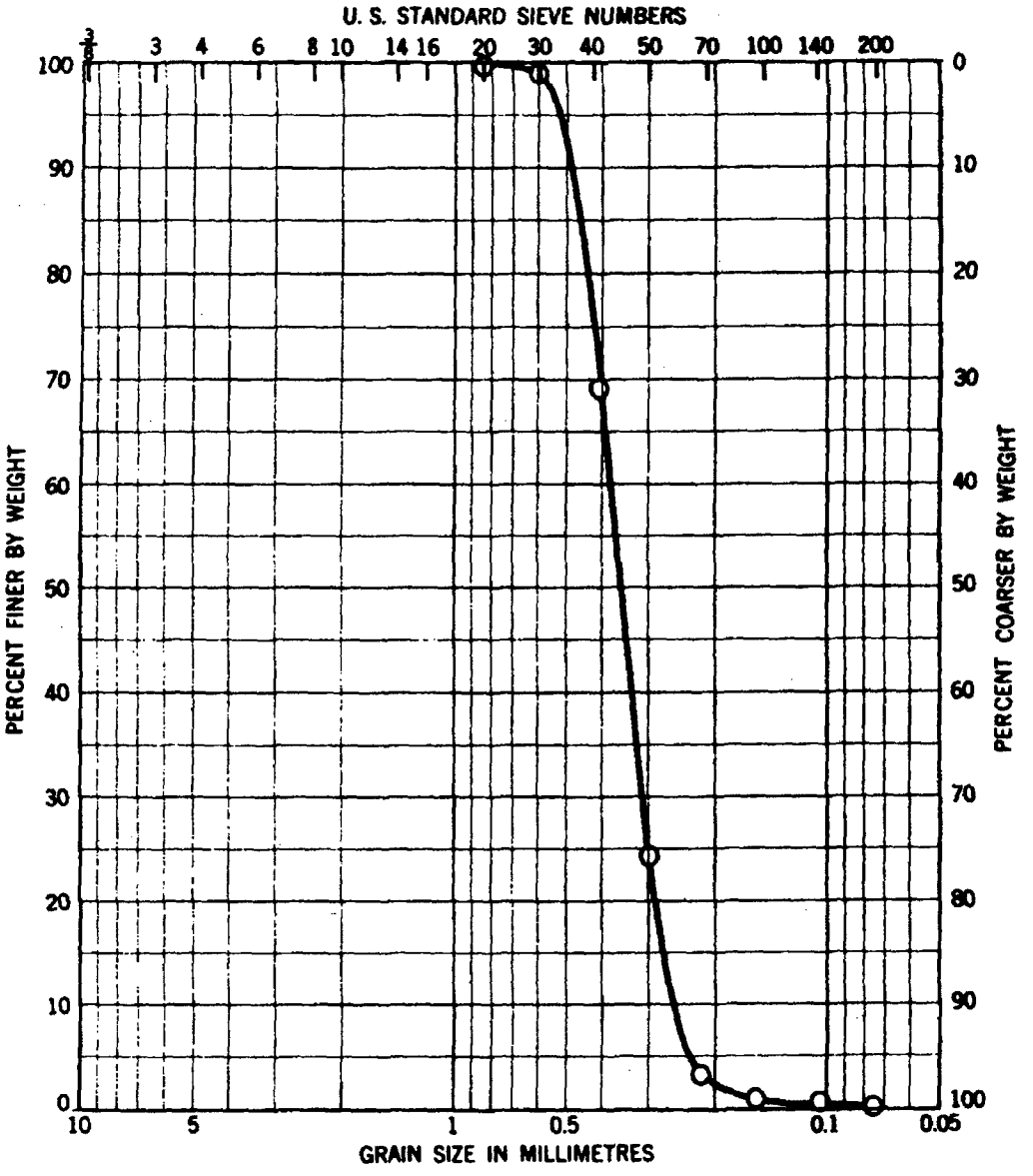
c. Effective Confining Pressure. The effective confining

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† H. B. Seed, and I. M. Idriss, "A Simplified Procedure for Evaluating Soil Liquefaction Potential," Journal of Soil Mechanics and Foundations Engineering, ASCE, Vol 97, No. SM9, Sep 71.

†† Available from Lone Star Company, 9315 San Leandro St., Oakland, Calif. 94603. Small quantities available from C. K. Chan, Richmond Field Station, University of California, Berkeley, Richmond, Calif., while supplies last. An alternative sand and corresponding calibration curve available from WES Soils Research Center, Vicksburg, Miss. 39180.

‡ Mulilis et al., op. cit.; Silver, op. cit.; and Silver et al., op cit.



PHYSICAL PROPERTIES

$\gamma_{d\ MAX}$ = 105.7 PCF e_{MIN} = 0.56

$\gamma_{d\ MIN}$ = 89.3 PCF e_{MAX} = 0.85

G_s = 2.65 D_{50} = 0.36 MM

Figure 11. Grain-size distribution and physical properties of Monterey No. 0 sand (SP)

pressure to be used is 14.50 psi. Sufficient back pressure to achieve a B value greater than 0.95 is required.

d. Testing Frequency. 1 Hz.

e. Test Results. Figure 9 presents the range in acceptable values for stress ratio versus number of cycles to initial liquefaction and 10 percent double amplitude strain.

8. POSSIBLE ERRORS. In addition to those described in paragraph 9, Appendix X, TRIAXIAL COMPRESSION TESTS, the following are possible errors that would cause inaccurate cyclic triaxial strength determinations :

a. Apparatus. (1) Loading wave form. Nonuniform, eccentric load wave forms may result from excessive piston friction, improper gain setting, or operation of the closed-loop servo valve. Insufficient air or hydraulic fluid at failure conditions will cause unacceptable load reduction. Misalignment between the loading piston and load actuator may also cause unacceptable loading wave forms.

(2) Electronic transducers and recording equipment. Improper calibration or sensitivity of the electronic transducers, incorrect balancing of amplifiers or zero settings, or improper range settings will result in inaccurate recording of actual loads, deformations, and pressures occurring during the test. It is also essential that the recorder response be rapid enough to follow all changes in the transducer output.

b. Specimen Preparation and Testing. (1) Specimen dimensions not measured precisely or density improperly calculated. A circumferential tape for measuring specimen diameter is recommended for obtaining precise measurements. Twice the thickness of the membrane must be subtracted for measurements of single membrane-encased specimens. An improperly calibrated burette will lead to incorrect volume change measurements during consolidation with resulting errors in specimen density computations.

(2) Percent undercompaction in lower specimen layers improper for achieving uniform density.

(3) Incomplete saturation resulting in low B values. A variety of problems can cause low B values: (a) use of insufficiently de-aired water may prevent dissolving of air in the specimen without resorting to extremely high back pressures; (b) incomplete de-airing or saturation of pore pressure transducer and drainage lines (can be avoided by applying a vacuum); and (c) system leaks due to punctured membrane, poor membrane sealing to cap and base, loose fittings, or improperly designed O-ring grooves (can be detected by using a bubble chamber while applying vacuum to the system).

(4) Incorrect application of consolidation stresses. Whenever the loading piston is fixed to the specimen cap, the static uplift load equal to the area of the piston rod multiplied by the chamber pressure must be accounted for when applying pressures, whether during back-pressure saturation, B-value checks, or consolidation.

(5) Prestraining specimens. By allowing the effective confining stress to vary, specimen strength can be greatly increased.†

(6) Scale factor for recorder traces not in agreement with actual data measurements. Reduction of data from recorder traces where incorrect scale factor, i.e., inches on recorder trace per pound (load), per inch (deformation), or per pounds per square inch (pressure), is used will result in data reduction error. The calibration steps should always be recorded on the recorder trace prior to and after testing.

† Townsend, op. cit.

CYCLIC TRIAXIAL TEST (SPECIMEN DATA)			Date
Project			Boring No.
Sample No.	Depth El.	Test No.	Method of Specimen Preparation

Initial Condition of Specimen

Membrane thickness, in.	T	Top		Center		Bottom		Avg	
Diameter, in.	D_o	Top		Center		Bottom		Avg	
Height, in.	H_o	North	East	South	West	Avg			
Area, sq in. = $0.7854\ D_o^2$	A_o		Dry density, lb/cu ft = $(W_s/V_o) \times 62.4$	γ_{do}					
Volume, cc = $16.39\ A_oH_o$	V_o		Relative density, %	D_{do}					
Initial weight soil, g	W_s		Specific gravity	G_s					

Condition of Specimen After Saturation and Consolidation

After Saturation			After Consolidation		
Change in height, in.	ΔH_{sat}		Change in height, in.	ΔH_c	
Height, in. = $H_o - \Delta H_{sat}$	H_{sat}		Height, in. = $H_{sat} - \Delta H_c$	H_c	
Area, sq in.	A_{sat}		Change in volume, cc	ΔV	
Volume, cc, = $16.39A_{sat}H_{sat}$	V_{sat}		Volume, cc = $V_{sat} - \Delta V$	V_c	
			Area, sq in. = $0.061\ V_c/H_c$	A_c	
			Void ratio	e_c	
			Dry density, lb/cu ft = $(W_s/V_c) \times 62.4$	γ_{dc}	
			Relative density, %	D_{dc}	

Equations

$$D_{avg} = \frac{D_{top} + D_{center} + D_{bottom}}{3} - 2T$$

$$A_{sat} = \frac{A_o}{H_o}(H_o - 2\Delta H_{sat})$$

$$e_c = \frac{G_s\gamma_w}{\gamma_{dc}} - 1$$

After Test Results

Consolidation Pressure, $\bar{\sigma}_{3c}$, psi		Consolidation Stress Ratio, $K_c = \bar{\sigma}_{1c}/\bar{\sigma}_{3c}$						
Tare No.		Percent	N	P_c, lb	P_E, lb	$\pm\sigma_{dc} = \frac{P_c + P_E}{2A_c}$, psi	$\pm\frac{\sigma_{dc}}{2\sigma_{3c}}$	
Tare and dry soil		$\epsilon_{da} = 5$						
Tare weight		$\epsilon_{da} = 10$						
Weight dry soil, g	W_s	ϵ_{da} =						
Remarks:		$\Delta u/\sigma_{2c} = 25$						
		$\Delta u/\sigma_{3c} = 50$						
		$\Delta u/\sigma_{3c} = 75$						
		$\Delta u/\sigma_{2c} = 100$						

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PLATE XA-1

CYCLIC TRIAXIAL TEST (SATURATION AND CONSOLIDATION WORKSHEET)			Date
Project			Boring No.
Sample No.	Depth El.	Test No.	Method of Specimen Preparation
Consolidation Pressure, $\bar{\sigma}_{3c}$, psi			Consolidation Stress Ratio, $K_c = \bar{\sigma}_{1c}/\bar{\sigma}_{3c}$

Computation of Saturation and Consolidation Data

Step	Chamber Pressure σ_3, psi	Uplift P_s lb	Back Pressure u, psi	Effective Confining Stress $\bar{\sigma}_{3c}$, psi	Axial Consolidation Load, $\bar{P}_s$, lb	Burette Reading cc	Dial Reading in.	Remarks

$P_s = \sigma_3 \times A_{rod}$ - wt of cap and piston $\qquad \bar{P}_s = P_s + A_{sat}(K_c - 1)\bar{\sigma}_{3c}$

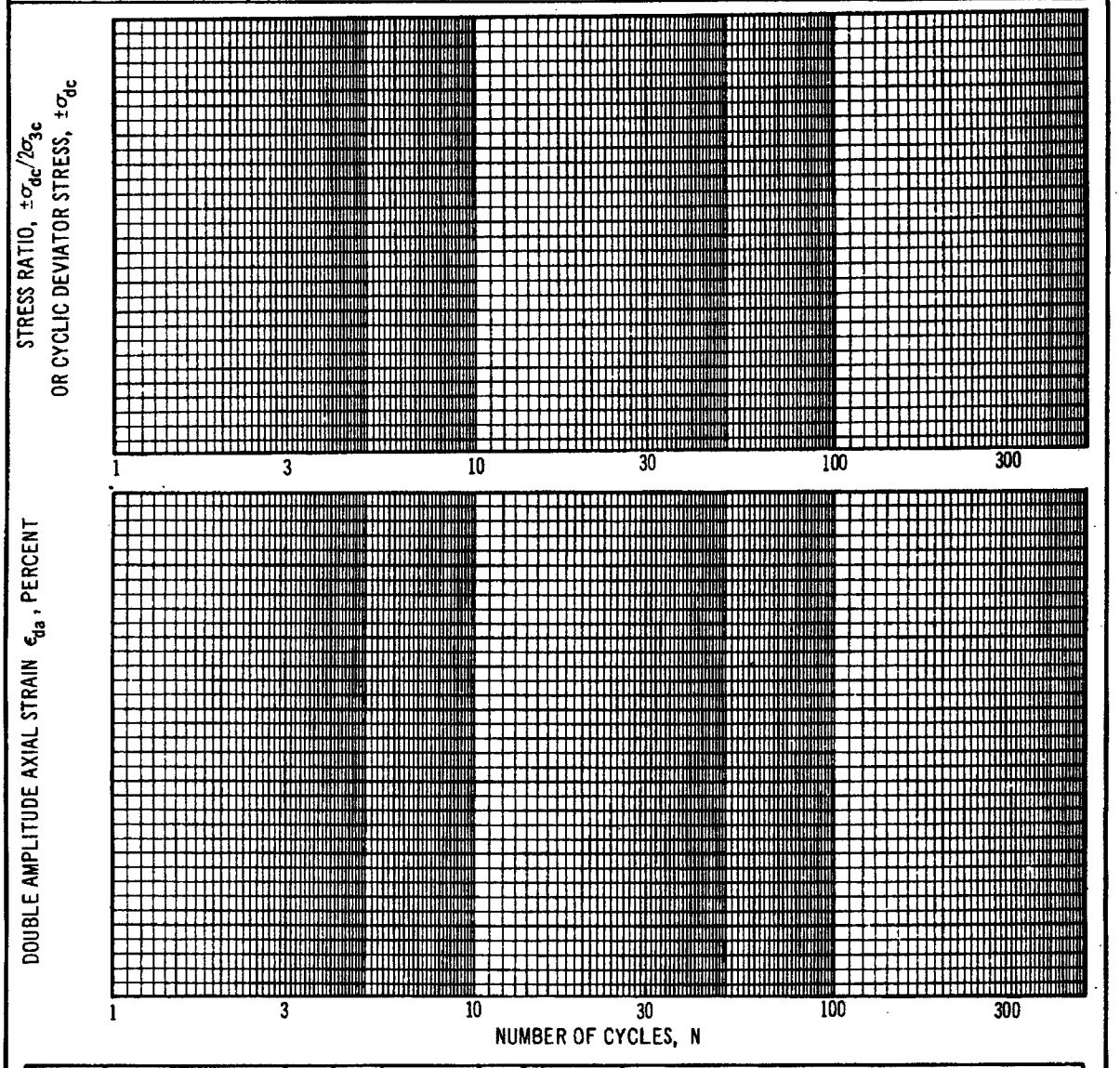
Desired Loading Conditions

Desired stress ratio, $\pm\sigma_{dc}/2\bar{\sigma}_{3c}$ =	
Cyclic dev stress, $\pm\sigma_{dc} = 2\bar{\sigma}_{3c} \times SR$ =	psi
Cyclic load, ΔP_c or ΔP_E, $\pm\sigma_{dc} \times A_c$	lb

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PLATE XA-1 (Cont'd)

CYCLIC TRIAXIAL TEST REPORT	Date
Project	Boring No.
Sample No.	Method of Specimen Preparation



SPECIMEN NO.	DENSITY	RELATIVE DENSITY	B	$\bar{\sigma}_{3c}$	CONSOLIDATION RATIO K_c	$\pm\sigma_{dc}$	$\pm\sigma_{dc}/2\bar{\sigma}_{3c}$	NUMBER OF CYCLES, N						
								PERCENT PORE PRESSURE INCREASE				PERCENT STRAIN, ϵ_{da}		
								N_{25i}	N_{50i}	N_{75i}	N_i	N_5	N_{10}	N_{20}

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PLATE XA-2

APPENDIX XB:

DETERMINATION OF CRITICAL VOID RATIO

1. **DEFINITION OF CRITICAL VOID RATIO.** Critical void ratio is that void ratio at which a cohesionless soil can undergo deformation or actual flow without volume change. In general, it is a function of confining pressure, i.e., an increase in confining pressure yields a lower value of critical void ratio. Theoretically, a soil with a void ratio above the critical value for the confining pressure is subject to flow failure or liquefaction if it undergoes sufficient undrained stress, whether that stress is cyclic or monotonic (steadily increasing). However, if the void ratio is lower than the critical value for the given confining pressure, the soil will not liquefy under monotonic stress increase. Liquefaction resulting from monotonic stress increase and "initial liquefaction" occurring under cyclic stresses are not the same phenomenon. For the purposes of this appendix, liquefaction refers to the behavior of a mass of cohesionless soil during flow slides. For monotonic loading conditions, failure is caused by a substantial reduction in shear strength due to large increases in pore pressure and the consequent great reduction of effective stress.

2. PRINCIPLES OF MONOTONIC TRIAXIAL TESTING. The monotonic stress-controlled triaxial compression test is used to determine the approximate values of critical void ratio over an applicable range of confining pressure. The test is comparable to an R test with pore pressure measurements (see Appendix X, TRIAXIAL COMPRESSION TESTS) with the exception that load is applied to the specimen in increments. The increments are arbitrary but should be chosen to produce several points on the stress-strain curve prior to peak deviator stress or the tendency to dilate, depending on the response of the

specimen as outlined below. The ability to execute an efficient test program to establish critical void ratio over a range of confining pressures requires recognition of the nature of specimen behavior. In general, three types of response from specimens can be anticipated, as illustrated in Figure 1.

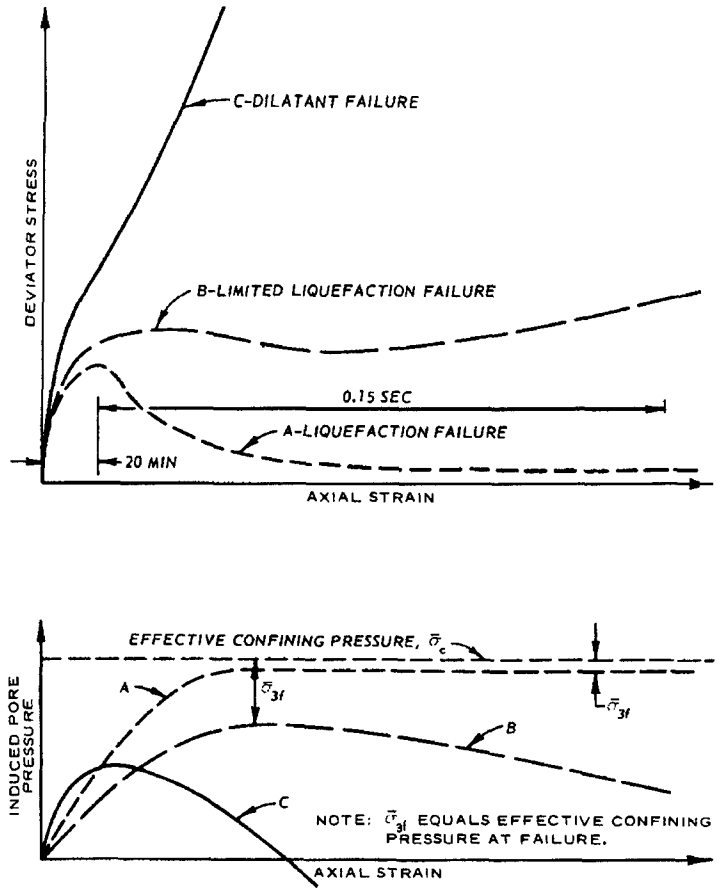


Figure 1. Typical stress-strain and pore pressure response, monotonic triaxial $\overline{R}$ tests on sands

a. Complete Liquefaction. If the void ratio after consolidation is sufficiently higher than the critical value for the given confining pressure, the specimen will bear several load increments with minor axial strain (usually less than 2 percent) while exhibiting steadily

accelerating pore pressure increases until failure by liquefaction occurs catastrophically at peak deviator stress. During the collapse of the specimen, which occurs over a fraction of a second, the deviator stress actually borne by the specimen declines rapidly while the pore pressure approaches, but not necessarily equals, the confining pressure.

b. Partial or Limited Liquefaction. If the void ratio after consolidation is slightly higher than or very nearly equal to the critical value for the given confining pressure, the specimen will tolerate several load increments with small axial strain until at peak deviator stress sustained deformation will occur. The deviator stress will essentially remain constant or suffer minor decline during the deformation. In some cases, deformation will cease, but the addition of small increments of load that counter the effects of area increase due to strain will maintain deformation. During the deformation, the pore pressure will rise to a value somewhat less than the confining pressure. In the latter stages of the test, the specimen response may become dilative, i.e., it will bear additional deviator stress, and the pore pressure will begin to decrease.

c. Dilatant Failure. If the void ratio after consolidation is lower than the critical value for the given confining pressure, the specimen will tend to dilate, i.e. continue to accept loading, and the pore pressure decreases.

3. APPARATUS. All the equipment is listed in Appendix X, TRIAXIAL COMPRESSION TESTS, for performing triaxial compression R tests with pore pressure measurements plus (a) electronic transducers and high-speed recorders for data acquisition, and (b) stress-controlled loading capacity.

a. Loading Devices. The equipment may be either (1) a pneumatic system, (2) a closed-loop electrohydraulic system, or (3) a deadweight system. A deadweight system is practical only for 1.4-in.-diameter specimens. Such small specimens may be used, but

they are undesirable from the standpoint of specimen preparation (see Appendix X, TRIAXIAL COMPRESSION TESTS, pp X-15 and X-16). Figure 2 shows a satisfactory arrangement for deadweight loading, which consists of a horizontal loading bar bearing on the load rod (piston) and connected to a weight hanger suspended beneath the triaxial chamber. A stopblock must be provided beneath the weight hanger that will permit deformation of the specimen equivalent to not less than 30 percent axial strain but will prevent contact of the horizontal loading bar with the top of the triaxial chamber. The loading assembly should be designed to support 300 lb and constructed of lightweight material. Pneumatic or closed-loop electrohydraulic systems must be capable of maintaining the axial load, i.e., supply sufficient air or oil to allow the piston to follow the sudden and rapid deformation during liquefaction.

b. Specimen Cap. The specimen cap should be of a lightweight noncorrosive material equipped with porous metal or porous stone inserts and drainage connections. The cap can be similar to those in Figure 4 of Appendix X, TRIAXIAL COMPRESSION TESTS, but lubricated end platens similar to those used by Castro† are more desirable . Preferable specimen diameter is 2.8 in. or larger.

c. Triaxial Compression Chamber. The chamber is the same as that in paragraph 3b of Appendix X, TRIAXIAL COMPRESSION TESTS, or the newer design shown in paragraph 3c of Appendix XA, CYCLIC TRIAXIAL TESTS, except that the rigid connection of the loading piston to specimen cap is not required.

d. Loading Piston to Loading Equipment Connection. See paragraph 3d of Appendix XA, CYCLIC TRIAXIAL TESTS.

e. Recording Equipment. See paragraph 3e of Appendix XA, CYCLIC TRIAXIAL TESTS.

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† G. Castro, " Liquefaction of Sands," Harvard Soil Mechanics Series No. 81, Jan 1969, Cambridge, Mass.

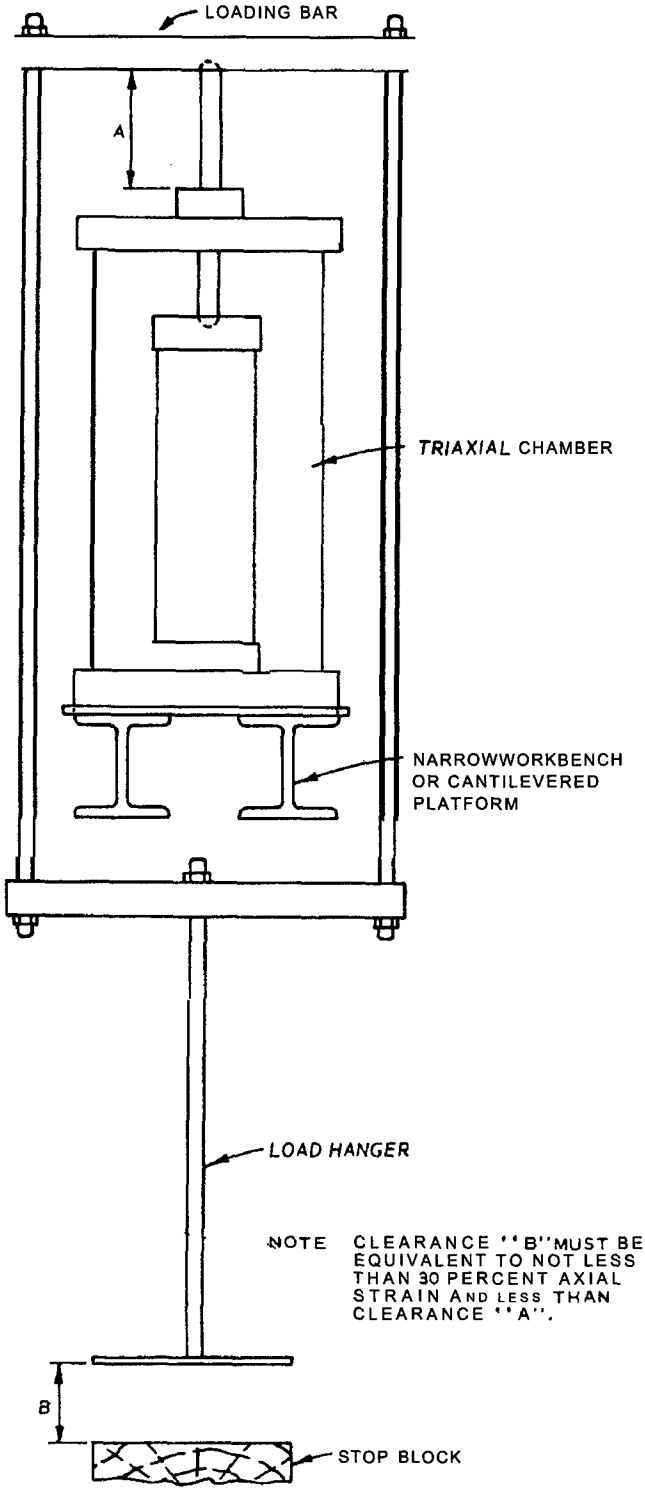


Figure 2. Deadweight loading system

f. Measurement Transducers. See paragraph 3f of Appendix XA, CYCLIC TRIAXIAL TESTS.

g. Back-Pressure Saturation Equipment. See paragraph 3g of Appendix XA, CYCLIC TRIAXIAL TESTS.

h. Tamping Rod for Moist Tamping Specimen Preparation. (Optional) See paragraph 3h of Appendix XA, CYCLIC TRIAXIAL TESTS.

4. TESTING PROCEDURES. Specimen preparation, back-pressure saturation, and consolidation procedures specified in paragraphs 4b and 7b(1) through 7b(7) of Appendix X, TRIAXIAL COMPRESSION TESTS, are adequate and pertinent. However, preferred techniques for handling undisturbed samples of cohesionless soils, preparing remolded specimens, and anisotropic consolidation are as follows:

a. Reconstituted Specimen Preparation (Moist Tamping). The procedure is as outlined in paragraph 4a of Appendix XA, CYCLIC TRIAXIAL TESTS, except that the initial water content of the material should be near the bulking water content, i.e., 5-10 percent. This water content permits molding of specimens in the vicinity of the critical void ratio, which is typically a relatively high value, i.e ., usually between 10 and 40 percent relative density, depending on the confining pressure.

b. Specimen Preparation (Undisturbed). See paragraph 4b of Appendix XA, CYCLIC TRIAXIAL TESTS.

c. Specimen Measurement. Inasmuch as void ratio is a principle of the test results, it is imperative that accurate initial specimen dimension measurements and volume change determinations during saturation and consolidation be made. Diameter measurements should be made to the nearest 0.001 in. using a circumferential tape†

† Commercially available from PI Tape, Box 398, Lemon Grove, Calif. 92045.

at the top, midheight, and bottom of the specimen or dial-gage calipers at the same locations, except two readings should be taken at each position by rotating the dial-gage calipers 90 deg to produce a total of six diameter readings. Care must be taken not to deform or disturb the relatively loose specimen during diameter measurements. Height measurements to the nearest 0.001 in. at four locations and weights to the nearest 0.1 g are recommended for 2.8-in.-diameter specimens. Record the data on the data sheet. (Plate XB-1).

d. Preliminary Seepage Saturation. See paragraph 4d of Appendix XA, CYCLIC TRIAXIAL TESTS.

e. Consolidation. See paragraph 4e of Appendix XA, CYCLIC TRIAXIAL TESTS.

f. Monotonic Loading. If it does not already exist, a large air pocket should be formed at the top of the triaxial chamber by draining water† from the chamber but leaving enough water to cover the top of the specimen. The air pocket is required, so that the large rapid piston movements into the chamber at the onset of failure do not create chamber pressure fluctuations. The procedure is as follows:

(1) Record test number and specimen identification on recorder trace.

(2) Zero the recorder and transducer outputs, and record calibration steps and scale factors.

(3) Close valve F (D should have already been closed) as shown in Figure 16 of Appendix X, TRIAXIAL COMPRESSION TESTS.

(4) Initiate incremental axial loading using an increment

† Compressed air instead of water may be used as the confining medium provided saturation and consolidation procedures do not exceed 8 hr.

determined as described in (5) below. During loading, the chamber pressure is maintained constant, and the axial load, axial deformation, and change in pore water pressure are recorded with time. Slow recorder speeds can be used as long as the specimen is deforming slowly; manual recording of load, pore water pressure, and deformation can be made as long as specimen behavior permits (this practice can save time in data reduction in that recorder traces are troublesome to read). Loading is continued on approximately 1-min intervals until either (a) the specimen liquefies or partially liquefies, or (b) the specimen tends to dilate. A complete stress-strain record, such as shown for liquefaction or limited liquefaction in Figure 1, is difficult to obtain and requires experience- However, it is achieved by increasing the recorder speed to 10 to 20 in. of record per second just as the specimen indicates impending failure. The clues to approaching rapid failure are an obvious increase in deformation rate and steadily accelerating increases in pore water pressure. If the specimen deforms steadily but not catastrophically, additional load increments may be applied as the straining tends to cease until the pore pressure begins to decline.

(5) The magnitude of the monotonic load increments is arbitrary but should be selected to provide several points on the stress-strain curve prior to peak deviator stress. For a confining pressure, $\bar{\sigma}_{3c}$, of 0.5 tsf, a reasonable increment would be equivalent to about 1.0 psi of deviator stress based on the initial area of the specimen. If the confining pressure is 5.0 tsf, an increment equivalent to about 5.0 psi is more practical. In any case, it is necessary to reduce the load increment as peak deviator stress is approached in order to prevent the application of the last increment from significantly exceeding the true peak value. Confident use of such a procedure must be learned through experience with initial tests on a given soil.

g. Specimen Removal. Following testing, the specimen should

be carefully removed from the triaxial cell to make sure that no particles are lost, then dried and weighed for dry unit weight calculations.

5. COMPUTATIONS. a. From the initial specimen data recorded on the data sheet (Plate XB-1), compute the initial water content, initial void ratio, initial dry density, and if required, the initial relative density, using equations presented in Appendixes I, WATER CONTENT, and II, UNIT WEIGHTS, VOID RATIO, POROSITY, AND DEGREE OF SATURATION.

b. From saturation data, compute the values of Skempton's pore pressure parameter B, and record any specimen height changes during saturation, the static piston loads required to counterbalance uplift pressures, and the magnitude of applied back pressure. If the specimen has suffered a change in height during saturation, correct the initial volume assuming the initial area remains valid.

c. Record specimen dimension changes during consolidation, and compute the area of specimen after consolidation (A_c), dry density after consolidation (γ_d), and the void ratio after consolidation as follows or as indicated in Plate XB-1:

$$A_c = \frac{V_o - \Delta V}{H_o - \Delta H}$$

where,

V_o = initial volume of specimen, cc

ΔV = change in volume of water in specimen during consolidation, from burette reading

H_o = initial height of specimen, cm

ΔH = change in height during consolidation, cm

6. PRESENTATION OF RESULTS. The critical void ratio is estimated from a plot of void ratio after consolidation, e_c, versus the effective confining pressure at failure, $\bar{\sigma}_{3f}$, for specimens that exhibit complete or limited liquefaction. For specimens that exhibit complete

or limited liquefaction, $\bar{\sigma}_{3f}$ is taken as the difference between the consolidation confining pressure, $\bar{\sigma}_{3c}$, and the maximum pore water pressure generated during the test (see Figs. 1 and 3). Figure 4 illustrates construction of the critical void ratio, which is the approximate relationship between e_c and $\bar{\sigma}_{3f}$ for the range of σ_{3c} The most desirable test series includes two to three tests at each of several pertinent confining pressures, σ_{3c}; e.g., 0.5, 1.0, 2.0, 4.0, and 8.0 tsf. The objective is to build the specimens near the critical void ratio, but slightly above it, to obtain complete and limited liquefaction in failure. This can be accomplished by trial. A trial procedure is not wasteful since all tests are of value in establishing the critical void ratio relationship. It is suggested that a pilot test be performed with a specimen built to about 20 percent relative density and by applying a moderate consolidation stress (about 2 tsf). The behavior of this specimen should indicate the approximate position of critical void ratio. Smaller confining pressures would produce higher critical void ratios; and larger confining pressures, lower critical void ratios. Those specimens that tend to dilate upon loading serve to help fix a lower bound to the position of the e_c versus the $\bar{\sigma}_{3f}$ relationship.

7. POSSIBLE ERRORS. In addition to those described in paragraph 9 of Appendix X, TRIAXIAL COMPRESSION TESTS, the following are possible errors that would cause inaccuracies in the determination of critical void ratio.

a. Apparatus. (1) Loading system. Insufficient air or hydraulic fluid at failure conditions will cause unacceptable load reduction. Misalignment between the loading rod piston and load actuator or air piston may also cause unacceptable loading conditions.

(2) Electronic transducers and recording equipment. Improper calibration or sensitivity of the electronic transducers, incorrect balancing of amplifiers or zero settings, or improper range settings will result in inaccurate recording of actual loads, deformations, and

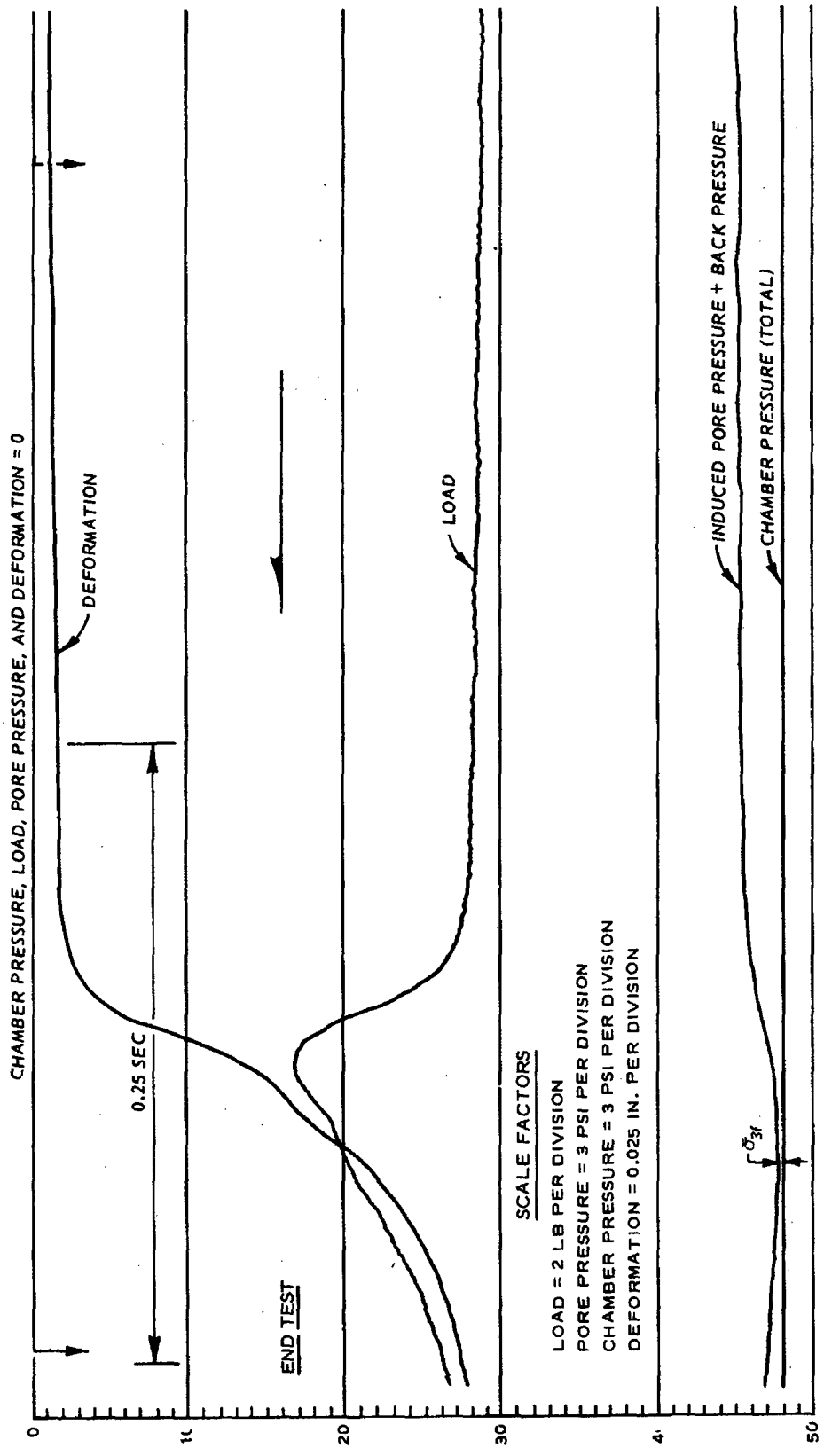


Figure 3. Facsimile of strip chart recorder readout for an anisotropically consolidated specimen that exhibited total liquefaction

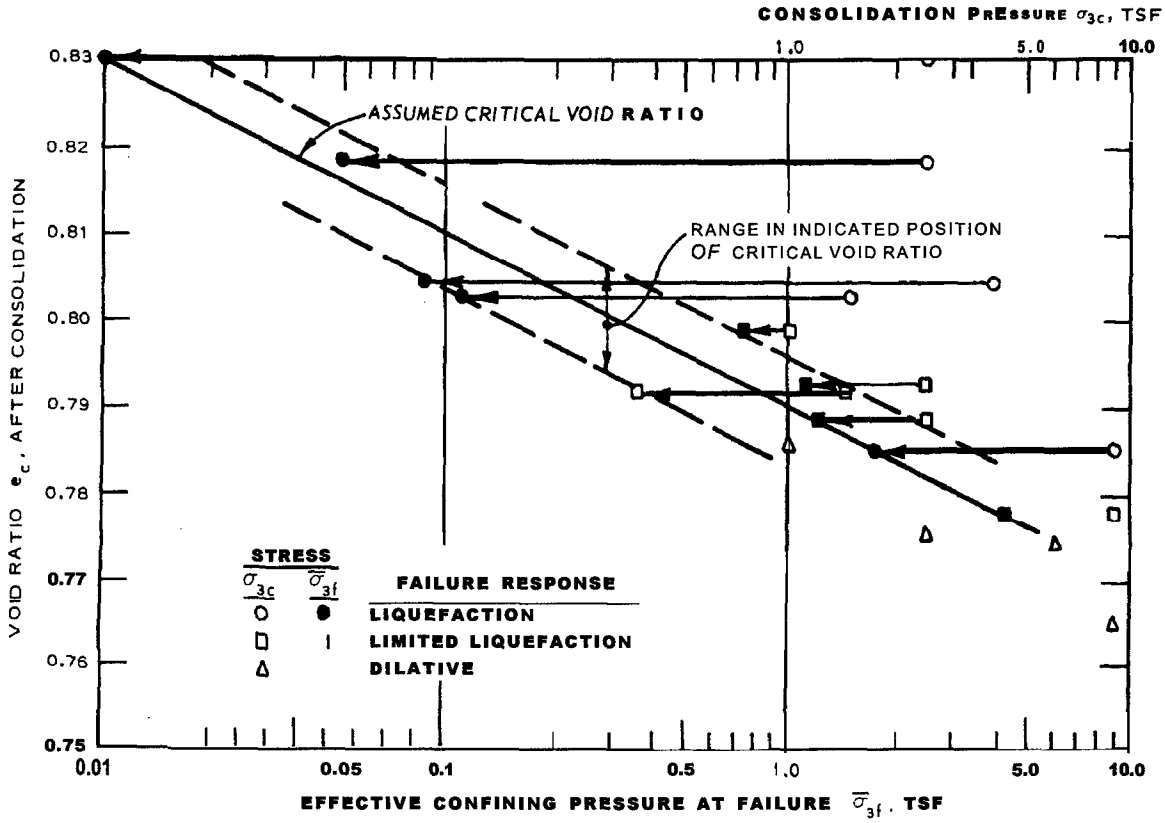


Figure 4. Example of the determination of approximate critical void ratio

pressures occurring during the test. It is also essential that the recorder response be rapid enough to follow all changes in the transducer output.

b. Specimen Preparation and Testing. (1) Specimen dimensions not measured precisely or density improperly calculated. A circumferential tape for measuring specimen diameter is recommended for obtaining precise measurements. Twice the thickness of the membrane must be subtracted for measurements of single membrane-encased specimens. An improperly calibrated burette will lead to incorrect volume change measurements during consolidation with resulting errors in specimen void ratio computations.

(2) Percent undercompaction in lower specimen layers improper for achieving desired average density.

(Uniform density in a reconstituted specimen is essential).

(3) Incomplete saturation resulting in low B values. The following problems can cause low B values: (a) use of insufficiently de-aired water may prevent dissolving of air in the specimen without resorting to extremely high back pressures; (b) incomplete de-airing or saturation of pore pressure transducer and drainage lines (can be avoided by applying a vacuum); and (c) system leaks due to punctured membrane, poor membrane sealing to cap and base, loose fittings, or improperly designed O-ring grooves (can be detected by using a bubble chamber while applying vacuum to the system).

(4) Assuming that the loading piston is not connected to the specimen cap, the static uplift load equal to the area of the loading piston multiplied by the chamber pressure must be accounted for when applying axial stresses.

(5) Scale factor for recorder traces not in agreement with actual data measurements. Reduction of data from recorder traces where scale factor, i.e., inches on recorder trace per pound (load), per inch (deformation), or per pounds per square inch (pressure), will result in data reduction error. The calibration steps should always be recorded on the recorder trace prior to and after testing.

APPENDIX XI:

UNCONFINED COMPRESSION TEST

1. INTRODUCTION. The unconfined compression test is used to measure the unconfined compressive strength of a cohesive soil. The unconfined compression test is applicable only to coherent materials such as saturated clays or cemented soils that retain intrinsic strength after removal of confining pressure; it is not a substitute for the Q test. Dry or crumbly soils, fissured or varved materials, silts, and sands cannot be tested meaningfully in unconfined compression. in this test, a laterally unsupported cylindrical specimen is subjected to a gradually increased axial compression load until failure occurs. The unconfined compression test is a form of triaxial test in which the major principal stress is equal to the applied axial stress, and the intermediate and minor principal stresses are equal to zero. The unconfined compressive strength, q_u, is defined as the maximum unit axial compressive stress at failure or at 15 percent strain, whichever occurs first. The undrained shear strength, s_u, is assumed to be equal to one-half the unconfined compressive strength. The axial load may be applied to the specimen either by the controlled strain procedure, in which the stress is applied to produce a predetermined rate of strain, or by the controlled stress procedure, in which the stress is applied in predetermined increments of load.

2. APPARATUS. The apparatus consists of the following:

a. Equipment for Preparing Specimen. A trimming frame as described% paragraph 3e of Appendix X, TRIAXIAL COMPRESSION TESTS, or a trimming cylinder with beveled cutting edges may be used for trimming specimens. The equipment should include wire saws and knives of various sizes and types for use with the trimming frame. A motorized soil lathe may be used advantageously under certain circumstances. A miter box or cradle is required to trim the specimen to a fixed length and to ensure that the ends of the specimen are parallel with each other and perpendicular to the vertical axis of the specimen.

b. Loading Device . A number of commercially available controlled-strain or controlled-stress types of loading devices are suitable for applying the axial loads in the unconfined compression test. In

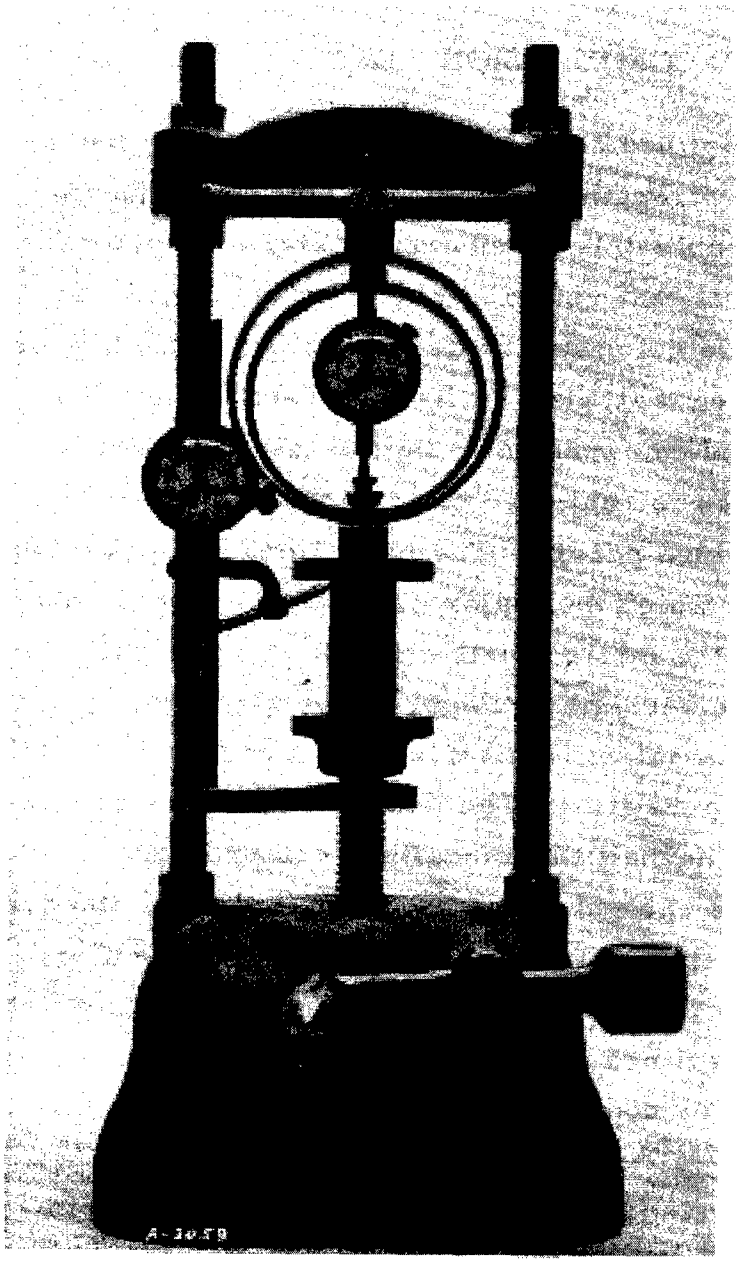


Figure 1. Typical unconfined compression test apparatus

general, controlled- strain type loading devices are preferable, and the procedures described herein are based on the use of this type of equipment. If available, an automatic stress-strain recorder may be used to measure and record applied axial loads and displacements. A typical loading device is shown in Figure 1. Any equipment used should be calibrated so that the loads actually applied to the soil specimen can be determined. The required sensitivity of stress-measuring equipment for both controlled-stress and controlled-strain testing will vary with the strength characteristics of the soil. For relatively weak soils (compressive strengths less than 1.0 ton per sq ft), the unit load should be measurable to within 0.01 ton per sq ft. For soils with compressive strengths of 1.0 ton per sq ft or greater, the loads should be measurable to the nearest 0.05 ton per sq ft.

c. Measuring equipment, such as dial indicators and calipers, suitable for measuring the dimensions and axial deformation of a specimen

to the nearest 0.001 in.

d. Timing device, either a watch or clock with second hand.

e. Balances, sensitive to 0.1 g.

f. Other. Apparatus necessary to determine water content and specific gravity (see Appendixes I, WATER CONTENT - GENERAL, and IV, SPECIFIC GRAVITY).

3. PREPARATION OF SPECIMENS. a. Specimen Size. Unconfined compression, specimens shall have a minimum diameter of 1.0 in. (preferably 1.4 in.), and the largest particle in any test specimen will be no greater than one-sixth the specimen diameter. The height-to-diameter ratio shall be not less than 2.1. Commonly used diameters of unconfined compression specimens are 1.4 and 2.8 in. Specimens of 1.4-in. diameter are generally used for testing cohesive soils which contain a negligible amount of gravel.

b. Undisturbed Specimens. Generally, undisturbed specimens are prepared from undisturbed tube or chunk samples of a larger size than the test specimen. Core or thin-wall tube samples of relatively small diameter may be tested without further trimming except for squaring the ends, if the condition of the soil requires this procedure. Specimens must be handled carefully to prevent remolding, changes in cross section, or loss of moisture. To minimize disturbance caused by skin friction between samples and metal sampling tubes, the tubes should be cut into short lengths before ejecting the samples. Sample ejection should be accomplished with a smooth continuous, and fairly rapid motion in the same direction that the sample entered the tube. All specimens shall be prepared in a humid room to prevent evaporation of moisture. The specimen shall be prepared as follows:

(1) From the undisturbed sample cut a section somewhat larger in length and diameter than the desired specimen size.

It is generally desirable to prepare duplicate specimens for unconfined compression testing, and selection of material for testing should be made with this in mind.

(2) Carefully trim the specimen to the required diameter using a trimming frame and various trimming tools (see Fig. 7, Appendix X, TRIAXIAL COMPRESSION TESTS). Remove any small shells or pebbles encountered during the trimming operations. Carefully fill voids on the surface of the specimen with remolded soil obtained from the trimmings. Cut the specimen to the required length, using a miter box (see Fig. 8, Appendix X, TRIAXIAL COMPRESSION TESTS). Where the presence of pebbles or crumbling results in excessive irregularity at the ends, cap the specimens with a minimum thickness of plaster of Paris, hydrostone, or other support material. Care must be taken to insure that the ends of the specimen are parallel with each other and perpendicular to the vertical axis of the specimen.

(3) From the soil trimmings obtain 200 g of material for specific gravity and water content determinations (see Appendixes I, WATER CONTENT - GENERAL, and IV, SPECIFIC GRAVITY).

(4) Weigh the specimen to an accuracy of ±0.01 g for 1.4-in.-diameter specimens and ±0.1 g for 2.8-in.-diameter specimens. If specimens are to be capped, they should be weighed before capping.

(5) Measure the height of the specimen with calipers or a scale and the diameter with calipers or circumference measuring devices. If the specimen is cut to a fixed length in a miter box, the length of the miter box can be taken as the height of specimen for routine tests, and additional height measurements are not usually necessary. It is always advisable to measure the diameter of the specimen after trimming, even though specimens are cut to a nominal diameter in a trimming frame. Make all measurements to the nearest ±0.01 in. Determine the average initial diameter , D_o, of the specimen using the diameters measured at the top, D_t, center, D_c, and bottom, D_b, of the specimen, as follows:

$$D_o = \frac{D_t + 2D_c + D_b}{4}$$

(6) If the specimen is not tested immediately after preparation, precautions must be taken to prevent drying and consequent development of capillary stresses. When drying before or during the test is anticipated, the specimen may be covered with a thin coating of grease such as petrolatum. This coating cannot be used if the specimen is to be used in a subsequent remolded test.

c. Remolded Specimens. Remolded specimens usually are prepared in conjunction with tests made on undisturbed specimens after the latter has been tested to failure. The remolded specimens are tested to determine the effects of remolding on the shear strength of the soil. The remolded specimen should have the same water content as the undisturbed specimen in order to permit a comparison of the results of the tests on the two specimens. The remolded specimen shall be prepared as follows:

(1) Place the failed undisturbed specimen in a rubber membrane and knead it thoroughly with the fingers to assure complete remolding of the specimen. Take reasonable care to avoid entrapping air in the specimen and to obtain a uniform density.

(2) Remove the soil from the membrane and compact it in a cylindrical mold with inside dimensions identical with those of. the undisturbed specimen. The compaction effort is not critical since the water contents of soils subjected to remolded tests are always considerably wetter than optimum. Care must be taken, however, to insure uniform density throughout the specimen. A thin coat of petrolatum on the inside of the molding cylinder will assist in the removal of the specimen after compaction.

(3) Carefully remove the specimen from the mold, preferably by means of a close fitting piston, and plane off the top of the specimen. The specimen is then ready for testing.

(4) Follow the steps outlined in paragraphs 3b(4) and 3b(5).

4. PROCEDURE. The procedure shall consist of the following steps:

a. Record all identifying information for the sample such as project, boring number, visual classification, and other pertinent data on the data sheet (see Plate XI-1 which is a suggested form). The data sheet is also used for recording test observations described below.

b. Place the specimen in the loading device so that it is centered on the bottom platen; then adjust the loading device carefully so that the loading ram or upper platen barely is in contact with the specimen. If a proving ring is used for determining the axial load, contact of the platen and specimen is indicated by a slight deflection of the proving ring dial. Attach a dial indicator, sensitive to 0.001 in., to the loading ram to mea - sure vertical deformation of the specimen. Record the initial reading of the dial indicator on the data sheet (Plate XI-1). Test the specimen at an axial strain rate of about 1 percent per minute. For very stiff or brittle materials which exhibit small deformations at failure, it may be desirable to test the specimen at a slower rate of strain.. Observe and record the resulting load corresponding to increments of 0.3 percent strain for the first 3 percent of strain and in increments of 1 or 2 percent of strain thereafter. Stop the test when the axial load remains constant or when 20 percent axial strain has been produced.

c. Record the duration of the test, in minutes, to peak strength (time to failure), type of failure (shear or bulge), and a sketch of speci- men after failure on the data sheet (Plate XI-2).

d. After the test, place the entire specimen or a representative portion thereof in a container of known weight and determine the water content of the specimen in accordance with Appendix I, WATER CONTENT - GENERAL.

5. COMPUTATIONS. The computations consist of the following steps:

a. From the observed data, compute and record on the data sheet (Plate XI-1) the water content, volume of solids, void ratio, degree of

saturation, and dry density, using the formulas presented in Appendix II, UNIT WEIGHTS, VOID RATIO, POROSITY, AND DEGREE OF SATURATION.

b. Compute and record on the data sheet the axial strain, the corrected area, and the compressive stress, at each increment of strain by using the following formulas:

$$\text{Axial strain, } \varepsilon = \frac{\Delta H}{H_o}$$

$$\text{Corrected area of specimen, } A_{corr}, \text{ sq cm} = \frac{A_o}{1 - \varepsilon}$$

$$\text{Compressive stress, tons per sq ft} = \frac{P}{A_{corr}} \times 0.465$$

where

ΔH = change in height of specimen during test, cm
H_o = initial height of specimen, cm
A_o = initial area of specimen, sq cm
P = applied axial load, lb

6. **PRESENTATION OF RESULTS.** The results of the unconfined compression test shall be recorded on the report form shown as Plate XI-2. Pertinent information regarding the condition of the specimen, method of preparing the specimen, or any unusual features of each specimen (such as slickensides, stratification, shells, pebbles, roots, or brittleness) should be shown under "Remarks." The applied compressive stress shall be plotted versus the axial strain in Plate XI-2. The unconfined compressive strength, q_u, of the specimen shall be taken as the maximum or peak compressive stress. For tests continued to 20 percent strain without reduction of axial load occurring, the unconfined compressive strength as a rule shall be taken as the compressive stress at 15 percent strain.

Where the unconfined compressive strength of a specimen is also obtained after remolding, the sensitivity ratio, S_t, shall also be calculated and reported. The sensitivity ratio is defined as follows:

$$S_t = \frac{q_u \text{ (undisturbed)}}{q_u \text{ (remolded)}}$$

-7. POSSIBLE ERRORS. Following are possible errors that would cause inaccurate determinations of unconfined compressive strength:

a. Test not appropriate to type of soil.

b. Specimen disturbed while trimming.

c. Loss of initial water content. A small change in water content can cause a larger change in the strength of a clay, so it is essential that every care be taken to protect the specimen against evaporation while trimming and measuring, during the test, and when remolding a specimen to determine the sensitivity.

d. Rate of strain or rate of loading too fast.

a. USE OF OTHER TYPES OF EQUIPMENT FOR UNDRAINED SHEAR STRENGTH DETERMINATIONS. Various other types of laboratory equipment, such as cone penetrometers and vane shear apparatus, may be used advantageously in the laboratory as a supplement to the basic unconfined compression test equipment for determining the undrained shear strength of cohesive soils. The use of these testing devices generally results in savings in cost and time. However, the devices should be used with caution until sufficient data and procedural details are established to assure their successful application. Use of such testing apparatus, as a rule,

should be preceded by careful correlations with the results of tests with the basic unconfined compression test equipment on the same type of soil, and correlations developed for a given type of soil should not be used indiscriminately for all soils.

UNCONFINED COMPRESSION TEST

Date _____

Project _____

Boring No. _____ Sample No. _____

Specimen No.			Classification				
Tare No.			Height	H_o	in.	H_o	cm
Weight, g: Tare plus wet specimen			Average diameter	D_o	in.	D_o	cm
Tare plus dry specimen			Initial area	A_o	sq in.	A_o	sq cm
Water	W_w		Volume in cc = A_oH_o			V_o	
Tare			Volume of solids in cc = $W_s \div G_s$			V_s	
Wet specimen	W		Void ratio = $(V_o - V_s) \div V_s$			e_o	
Dry specimen	W_s		Saturation in % = $G_s w_o \div e_o$			S_o	%
Water content	w_o	%	Dry dens., lb/cu ft = $62.4\ (W_s \div V_o)$			γ_d	
Specific gravity of solids	G_s		Note: If only a portion of the specimen is used for the water content determination, $W_s = W \div \left(1 + \frac{w_o}{100}\right)$				

Elapsed Time min	Dial Reading 10^{-4} in.	Cumulative Change, ΔH 10^{-4} in.	Proving Ring Dial Reading 10^{-4} in.	Axial Load, P lb	Axial Strain $\epsilon = \Delta H/H_o$	$1 - \epsilon$	$A_{corr} = A_o/(1-\epsilon)$ sq cm	Compr Stress tons/sq ft $0.465(P/A_{corr})$

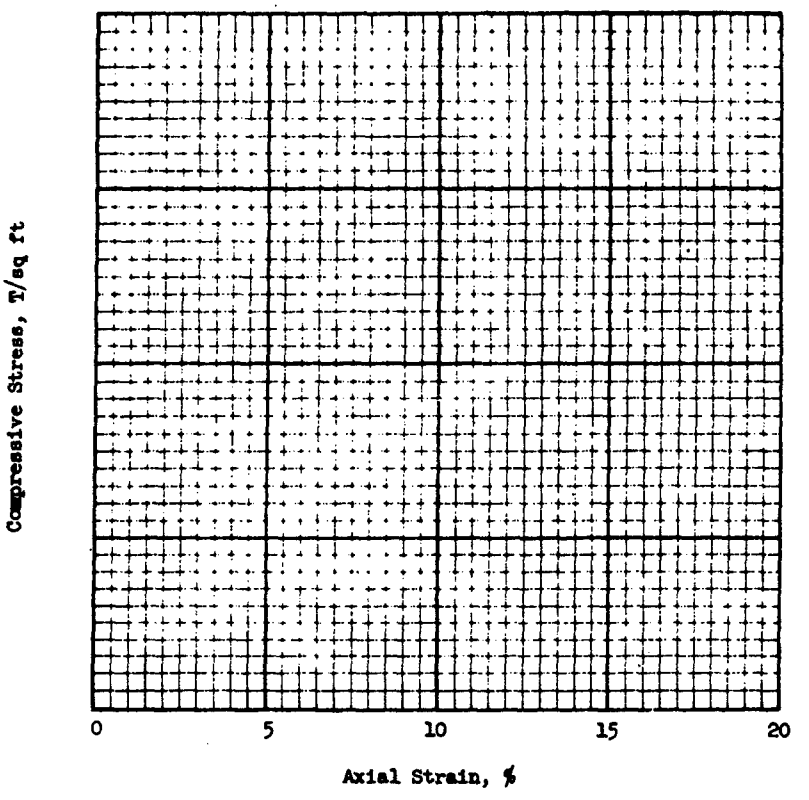
Remarks _____

_____ Type Failure _____

Technician _____ Computed by _____ Checked by _____

PLATE XI-1 ENG FORM 1 JUN 68 3857

Failure Sketches



☐ Controlled stress
☐ Controlled strain

Test No.						
Type of specimen						
Initial	Water content	w_o	%	%	%	%
	Void ratio	e_o				
	Saturation	S_o	%	%	%	%
	Dry density, lb/cu ft	γ_d				
Time to failure, min		t_f				
Unconfined compressive strength, T/sq ft		q_u				
Undrained shear strength, T/sq ft		s_u				
Sensitivity ratio		S_t				
Initial specimen diameter, in.		D_o				
Initial specimen height, in.		H_o				

Classification

LL	PL	PI	G_s

Remarks

Project

Area

Boring No. | Sample No.

Depth El | Date

UNCONFINED COMPRESSION TEST REPORT

ENG FORM 3659
1 JUN 65

PLATE XI-2

MONTONIC TRIAXIAL COMPRESSION TEST (SPECIMEN DATA)		Date
Project		Boring No.
Specimen No.	Method of Specimen Preparation	

			Before Test				After Test	
			Specimen	Trimmings			Specimen	
Tare No.								
Weight, g	Tare plus wet soil							
	Tare plus dry soil							
	Water	W_w		W_{wo}			W_{wf}	
	Tare							
	Wet soil	W						
	Dry soil	W_s						
Water content		w	%	W_o	%		W_f	%

Initial Condition of Specimen

Membrane thickness, in.	T	Top		Center		Bottom		Avg		
Diameter, in.		Top		Center		Bottom		Avg		
Height, in.	H_o	North		East		South		West		Avg
Area, sq in. = $0.7854\ D_o^2$	A_o			Void ratio = $(V_o - V_s) \div V_s$	e_o					
Volume, cc = $16.39\ A_o H_o$	V_o			Saturation, %	S_o					
Specific gravity	G_s			Dry density, lb/cu ft	γ_d					
Volume of solids, cc	V_s			Relative density, %	D_d					

Saturation

Chamber Pressure σ_3, psi	Uplift Load P_u, lb	Chamber Pressure Change $\Delta\sigma_3$, psi	Back Pressure psi	Pore Pressure u, psi	Pore Pressure Change Δu, psi	B

$$D_{avg} = \frac{D_{top} + D_{center} + D_{bottom}}{3} - 2T \qquad B = \Delta u / \Delta\sigma_3$$

$P_u = \sigma_3 \times A_{rod}$ - wt of piston

Condition of Specimen After Consolidation

Effective Confining Pressure After Consolidation, σ_c = _____ tsf

Change in height during consolidation, in.	ΔH		Area, sq in. = V_c/H_c	A_c	
Height, in. = $H_o - \Delta H$	H_c		Void ratio = $(V_c - V_s) \div V_s$	e_c	
Change in volume, cc	ΔV		Dry density, lb/cu ft	γ_d	
Volume, cc = $V_o - \Delta V$	V_c		Relative density, %	D_d	

FORM
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PLATE XB-1

APPENDIX XII:

RELATIVE DENSITY

1. GENERAL. Relative density expresses the degree of compactness of a cohesionless soil with respect to the loosest and the densest conditions of the soil that can be attained by specified laboratory procedures; a soil in the loosest state would have a relative density of 0 percent and in the densest state, of 100 percent. The dry unit weight of a cohesionless soil does not, by itself, reveal whether the soil is loose or dense, due to the influence of particle shape and gradation on this property. Only when viewed against the possible range of variation, in terms of relative density, can the dry unit weight be related to the compaction effort used to place the soil in an embankment or indicate the volume-change tendency of the soil when subjected to shear stresses. Originally, relative density, D_d, was defined by the equation

$$D_d, \text{ percent} = \frac{e_{max} - e}{e_{max} - e_{min}} \times 100$$

where

e_{max} = void ratio of the soil in the loosest state which can be attained in the laboratory

e = void ratio of the soil in place

e_{min} = void ratio of the soil in the densest state which can be attained in the laboratory

though, for ease of computation, relative density may be expressed in terms of dry unit weight by the equation

$$D_d, \text{ percent} = \frac{\gamma_d - \gamma_{d\ min}}{\gamma_{d\ max} - \gamma_{d\ min}} \times \frac{\gamma_{d\ max}}{\gamma_d} \times 100$$

where

γ_d = dry unit weight of the soil in place, called "in-place density"

$\gamma_{d\ min}$ = dry unit weight of the soil in the loosest state which can be attained in the laboratory, called "minimum density"

$\gamma_{d\ max}$ = dry unit weight of the soil in the densest state which can be attained in the laboratory, called "maximum density."

The in-place density of a soil may be determined by various methods in the field or from undisturbed samples brought into the laboratory; its determination will not be considered in this test procedure which covers only the methods for determining the minimum and maximum densities. There are few difficulties in determining minimum density by a standard method, but restrictions imposed by the availability of special apparatus under certain conditions or the character of the soil may not permit the determination of maximum density by a single, standard method to the exclusion of all others. However, the method described herein, employing a vibratory table, shall be preferred to all other methods;† the alternative method for attaining the maximum denseness of a soil by hammering on the mold (as described in Appendix XIIA, MODIFIED PROVIDENCE VIBRATED DENSITY TEST) should be followed only when the use of the vibratory table is not feasible. The method used for determining the maximum density must be reported when presenting relative density data.

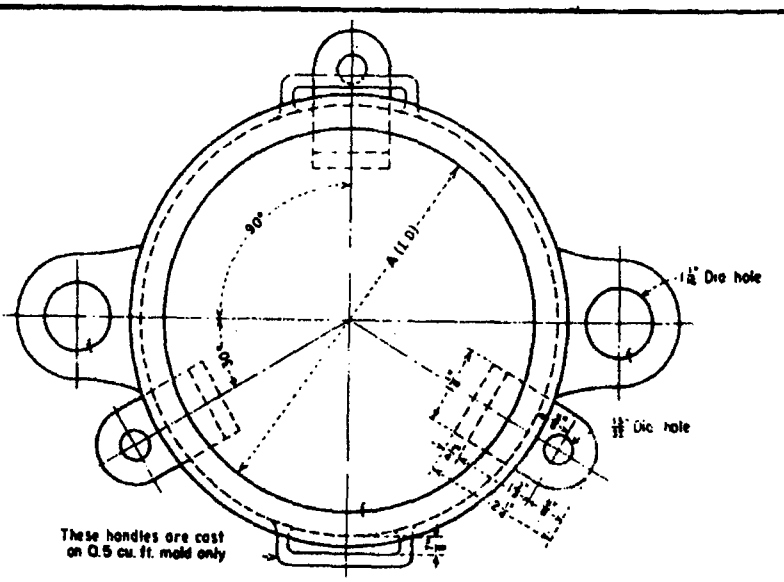
2. **APPARATUS.** The apparatus for determining the minimum and maximum densities of a cohesionless soil shall consist of the following:

a. Cylindrical molds or measures of 0.1-cu-ft capacity (6-in. inside diameter) and 0.5-cu-ft capacity (11-in. inside diameter), as shown in Figure 1.‡ The molds shouid be cast of silicon aluminum alloy ASTM-367 OA.

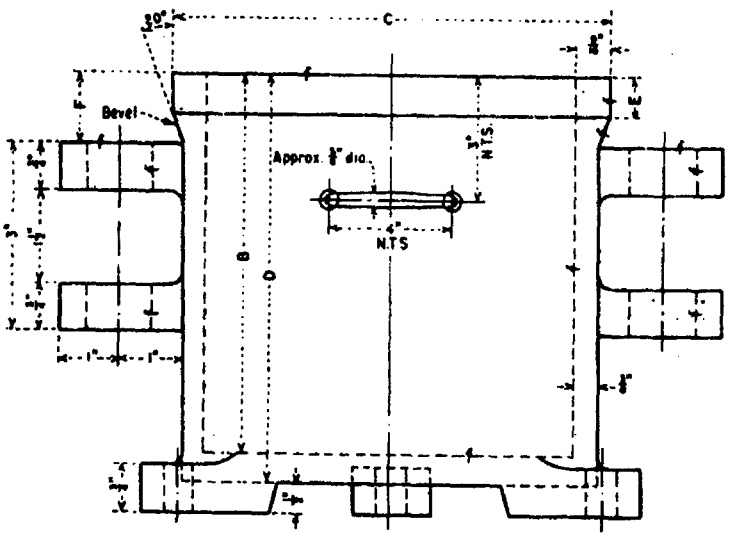
b. Surcharge assemblies, to fit each size mold, as shown in

† Other methods are currently being investigated to overcome the problem of segregation encountered in some soils using the vibratory table.

‡ These molds, as well as the surcharge assemblies and dial indicator holder shown in Figure 2, are available commercially from Stebbins Mfg. & Supply co., 1733 Blake St., Denver, Colo.



PLAN



ELEVATION

MOLD SIZE (CU FT)	DIMENSIONS					
	A	B	C	D	E	F
0.1	6.000"	6.112"	7⅜"	6½"	1"	1⅝"
0.5	11.000"	11.092"	12½"	9½"	⅝"	2"

Figure 1. Details of molds

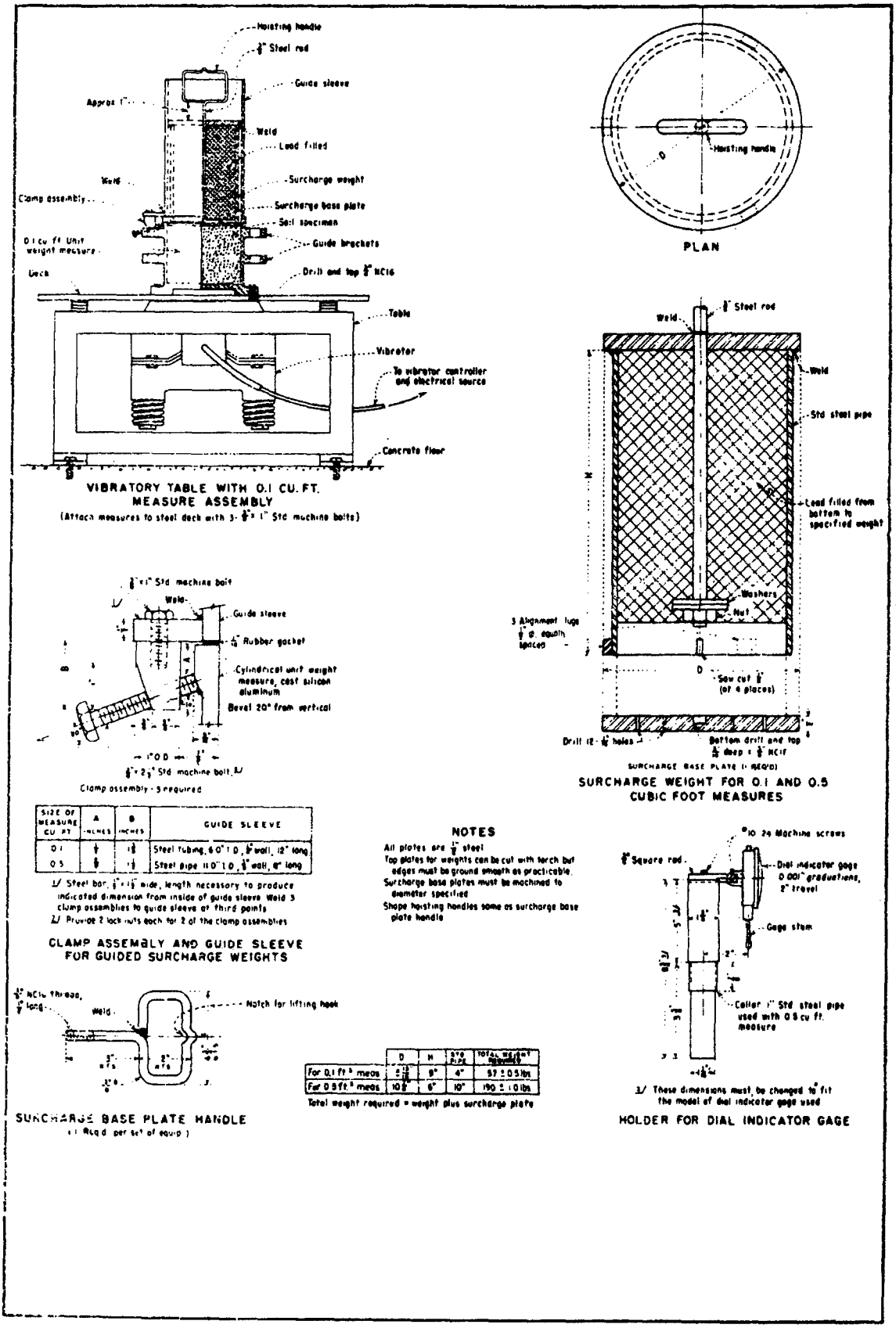


Figure 2. Equipment for maximum density determination

Figure 2. Each assembly shall include a surcharge weight (equivalent to 2 psi), a surcharge baseplate with handle and a guide sleeve with clamp assemblies.

c. Dial indicator, having 0.001-in. graduations and a 2-in. range.

d. Holder, for dial indicator, with collar, as shown in Figure 2.

e. Calibration bar, metal, 3 in. by 1/8 in. by 12 in. long.

f. Vibratory table, as shown in Figure 2, with a cushioned steel vibrating deck about 30 by 30 in., actuated by an electromagnetic vibrator and mounted to a concrete floor or slab of large mass. The vibrator should be a seminoiseless type with a net weight over 100 lb. The vibrator shall have a frequency of 3600 vibrations per minute and variable vibrator amplitudes to a maximum of at least 0.015 in. under a 250-lb load, and be suitable for use with 230-volt alternating current.†

g. Hoist, having a capacity of at least 300 lb, for handling the heavier surcharge weight.

h. Pouring device, as shown in Figure 3, which has metal funnels with 1/2- and 1-in.-diameter cylindrical spouts, each attachable to a metal can 6 in. in diameter by 12 in. high.

i. Hand scoop, large, metal.

j. Mixing pans, large, metal.

k. Sample splitter or riffle.

l. Straightedge, 15-in., steel.

m. Platform scales, having a capacity

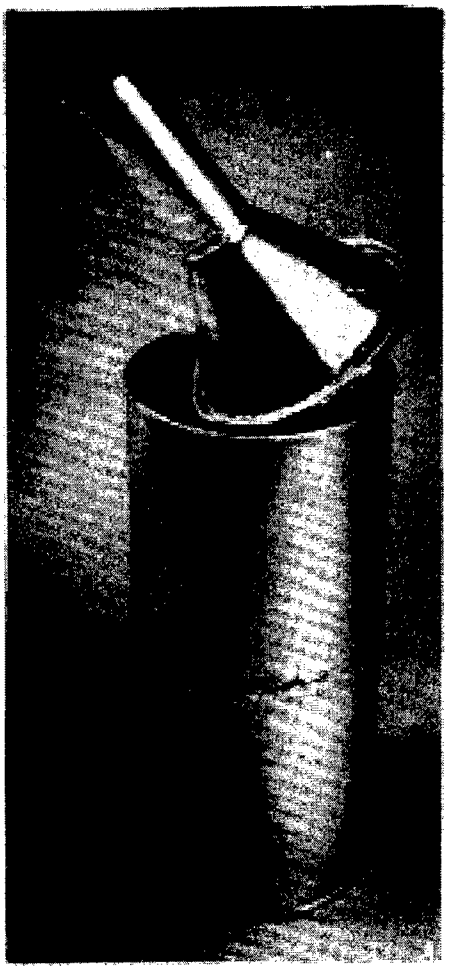


Figure 3. Pouring device, with i-in.-diameter spout

† The Syntron VP-80 and VP-240 Vibratory Packers, manufactured by Syntron Co., Homer City, Pa., have proven satisfactory. (The VP-80 has been replaced by a later model, VP-86, with the same characteristics.)

of at least 100 lb and sensitive to 0.01 lb.

n. Oven (see Appendix I, WATER CONTENT - GENERAL).

3. CALIBRATION OF EQUIPMENT. Each mold must be calibrated as follows :

a. Determine the weight, W_m, of each mold to the nearest 0.01 lb.

b. Determine the capacity, or total inside volume, V_m, of the 0.1 -cu-ft mold to the nearest 0.0001 cu ft and of the 0.5 -cu-ft mold to the nearest 0.001 cu ft.

c. Determine the inside cross-section area, A_m, at the open end of the 0.1-cu-ft mold to the nearest 0.0001 sq ft and of the 0.5-cu-ft mold to the nearest 0.001 sq ft.

d. Determine the initial dial reading, h_o, for each mold and surcharge baseplate combination in the following manner:

(1) Measure the thickness of the calibration bar, t_c, and of the surcharge baseplate, t_s, to the nearest 0.001 in.

(2) Lay the calibration bar across the top of the mold along the axis defined by the brackets for the dial indicator holder, as shown in Figure 4a, see page 9.

(3) Insert the dial indicator holder into the brackets on one side of the mold so that the dial indicator stem rests on the calibration bar, and note the dial reading. Then insert the dial indicator holder into the brackets on the opposite side of the mold and note the dial reading. The dial indicator holder should be placed in the same position in the guide brackets for each reading by means of matchmarks on the holder and on the brackets.

(4) Compute the average, h_r, of the two dial readings.

(5) Compute the initial dial reading, h_o, by the equation

$$h_o = h_r + t_c - t_s$$

4. PREPARATION OF SAMPLE. The soil to be tested must be oven-dried

and then permitted to cool in an airtight container. Aggregations of fine particles shall be thoroughly broken and a representative sample removed from the soil using a sample splitter or riffle. The representative sample should weigh at least 25 lb if the maximum particle size is less than 1-1/2 in. and at least 100 lb if the maximum particle size is between I -1/2 and 3 in. If the sample contains more than 10 percent by weight particles larger than 3 in. the determination of the maximum and minimum density becomes a test of a research nature.

5. PROCEDURE. Every precaution must be observed while handling the sample to prevent segregation and to preserve the oven-dried condition. The minimum and maximum densities shall be determined using the same sample, by first placing the soil into a mold in the loosest possible state to attain the minimum density and then vibrating it into the densest possible state to attain the maximum density. The 0.1-cu-ft mold shall be used for the determinations if the maximum particle size is less than 1-1/2 in. and the 0.5-cu-ft mold shall be used if the maximum particle size is between 1-1/2 and 3 in.

a. Minimum Density Determination. The procedure for determining the minimum density shall consist of the following steps:

(1) Record all identifying information for the sample. such as project, boring number, etc., on a data sheet (Plate XII-1 is a suggested form).

(2) Record the weight, inside volume, and end area of the mold on the data sheet.

(3) Carefully place the oven-dried soil into the mold in the loosest possible condition.† Fill the mold in layers, using the pouring device for material having maximum particle sizes less than 3/8 in. or the hand scoop for material having larger particle sizes. Exercise the greatest care at all times to avoid jarring the mold or otherwise disturbing the previously placed layers. When using the pouring device (with the

† If the maximum density of the oven-dried sample is to be determined also, the mold may be attached to the deck of the vibratory table before filling (see paragraph 5b(2)).

1/2 -in. -diameter spout if the maximum particle size passes the No. 4 sieve and the 1-in. -diameter spout if the maximum particle size is between the No. 4 sieve and 3/8-in. sieve), adjust the height of the spout to maintain a free fall of the soil of about 1 in. With a steady flow of soil from the spout, move the pouring device in a spiral path from the outside to the center of the mold to form each layer of uniform thickness without segregation.* When the maximum particle size of the sample exceeds 3/8 in., place the soil into the mold by means of the scoop held as closely as possible to the previously placed layer so the soil slides but does not fall from the scoop; restrain the larger particles with the hand where necessary to prevent their rolling from the scoop. Continue filling the mold until the soil rises slightly above the top of the mold, with care that no large particles which project above the top of the mold are placed in the final layer. Using the straightedge, carefully trim the soil surface level with the top of the mold.

(4) If the maximum density of the oven-dried sample is not to be determined,‡ weigh the mold and soil to the nearest 0.01 lb and record the weight on the data sheet; alternatively, the contents of the mold may be emptied into a mixing pan and weighed.

(5) Steps (3) and (4) should be repeated until consistent results (within 1 percent) are attained.

b. Maximum Density Determination With Oven-Dried Sample. The procedure for determining the maximum density shall consist of the following steps:

(1) Proceed in accordance with paragraphs 5a(1) through 5a(3).

(2) Attach the mold to the deck of the vibratory table, if this

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* Static electricity in dry sand can cause bulking similar to that produced by a trace of moisture on the particles; a static -eliminating balance brush can be applied to the equipment in contact with the sand when this effect becomes bothersome.

‡ If the maximum density of the oven-dried sample is to be determined also, proceed in accordance with paragraphs 5b(2) through 5b(7).

had not been done prior to filling the mold with soil (see paragraph 5a(3)).

(3) Place the guide sleeve on the top of the mold and clamp it firmly to the mold.* Lower the surcharge baseplate onto the surface of the soil and remove the handle. Using the hoist if necessary, lower the surcharge weight onto the surcharge baseplate.

(4) It has been determined that for a particular vibrating table, mold, and surcharge assembly, the maximum dry density of a specimen may be obtained at a displacement amplitude (rheostat setting) less than the maximum amplitude of which the apparatus is capable; i.e., dry density may increase with increase in rheostat setting to a setting, beyond which the dry density decreases. Therefore each laboratory should determine for its apparatus the rheostat setting at which maximum density is produced and use this setting for subsequent maximum density testing.‡ A clean, durable, subrounded to rounded material should be used in making these determinations (in both 0.1- and 0.5-cu-ft molds). The test should be performed as given in this appendix, except that the rheostat setting should be increased from zero to 100 in increments of 10 and measurements taken after each period of vibration; the material should be vibrated for a period of 8 min at each rheostat setting and not removed until after the last determination has been made (100 percent rheostat setting). The particle size distribution of the specimen should be determined before and after the test to assess the extent of degradation, if any.

(5) Remove the surcharge weight and guide sleeve from the mold, and obtain dial indicator readings on opposite sides of the surcharge

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* The inside surface of the guide sleeve must align with the inside surface of the mold, so two of the three clamping bolts should be provided with lock nuts, as noted in Figure 2. By properly adjusting and locking these two bolts, the guide sleeve will be drawn automatically into correct alignment when the third bolt is tightened.

‡ It may be desirable to redetermine the optimum rheostat setting at the inception of testing for each major project.

baseplate, as shown in Figure 4b. Record the dial readings on the data sheet.

(6) Remove the surcharge baseplate from the mold and detach the mold from the vibratory table.

(7) Weigh the mold and soil to the nearest 0.01 lb and record the weight on the data sheet; alternatively, the contents of the mold may be emptied into a mixing pan and weighed.

c. (Rescinded)

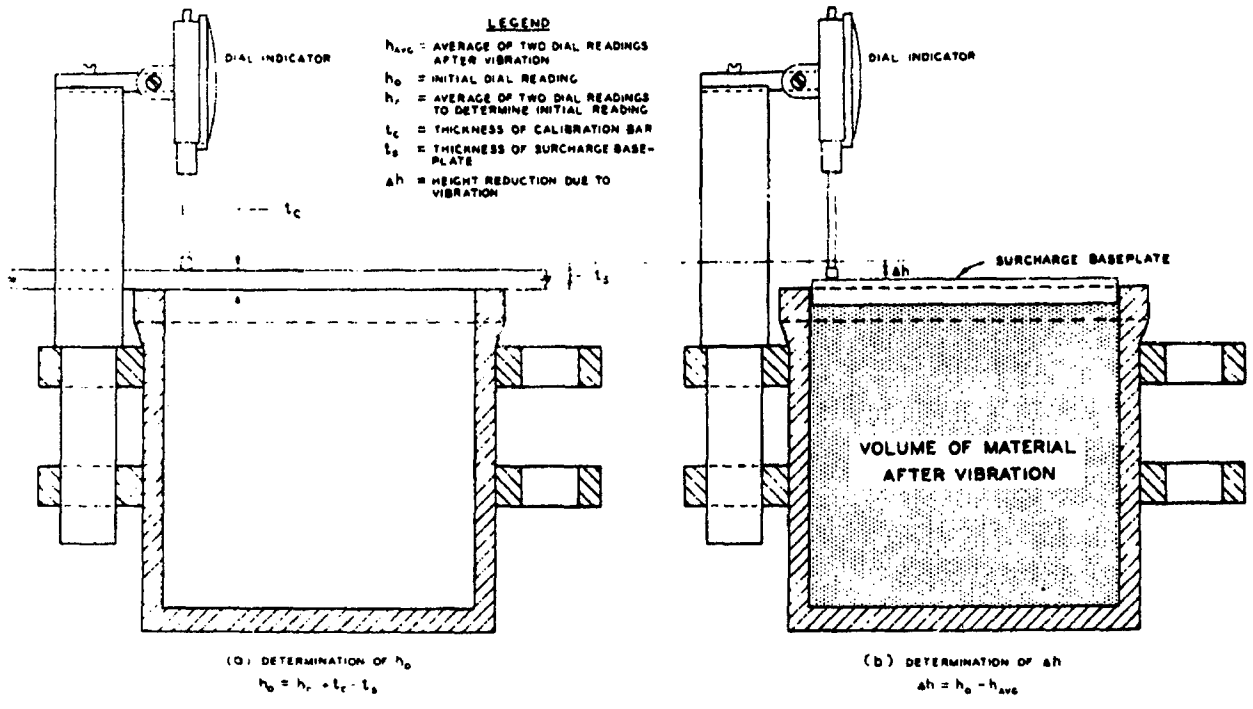


Figure 4. Determination of reduction in sample height due to vibration

6. COMPUTATIONS. The computations shall consist of the following steps:

a. Compute the weight of the dry soil, W_s, by subtracting the weight of the mold, W_m, (or mixing pan) from the weight of the mold (or mixing pan) and soil, W.

b. For the minimum density determination, the volume of the soil, V, is equal to the total inside volume of the mold, V_m.

c. For the maximum density determination, the volume of the soil, V, is equal to the total inside volume of the mold, V_m, minus the volume change, ΔV, caused by vibrating the soil. The volume change shall be computed in the following manner:

(1) Compute the average dial reading, h_{avg}, of the two readings taken after vibrating the soil

(2) Subtract the initial dial reading, h_o, from the average dial reading, h_{avg}, to obtain the height change, Δh.

(3) Compute the volume change by the equation

$$\Delta V = \frac{\Delta h}{12} \times A_m$$

where

ΔV = volume change, cu ft

Δh = height change, in.

A_m = end area of mold, sq ft

d. Compute the minimum and maximum densities to the nearest 0.1 lb per cu ft by the equation

$$\gamma_d = \frac{W_s}{V}$$

where

γ_d = dry density, lb per cu ft
W_s = weight of dry soil, lb
V = volume of soil, cu ft

e. If the in-place density of the soil is known, the relative density, D_d, can be computed by the equation given in paragraph 1, or determined graphically by means of Figure 5.

7. POSSIBLE ERRORS. Following are possible errors that would cause inaccurate determinations of relative density:

a. General. (1) Test not appropriate to type of soil. The relative density is meaningful only for cohensionless materials; if a soil has any appreciable dry strength, the methods for determining the minimum and maximum densities described in this Appendix are not applicable.

(2) Material segregated while being processed.

(3) Gain in moisture of oven-dried material before or during testing. A small amount of moisture in the soil can cause erroneous measurements of the minimum density and, to a much lesser degree, of the maximum density.

(4) Molds not accurately calibrated.

b. Minimum Density Determination. (1) Disturbance of mold during filling. Inadvertent jarring of the mold or impact of the falling particles will increase the measured minimum density.

(2) Segregation of material while filling mold.

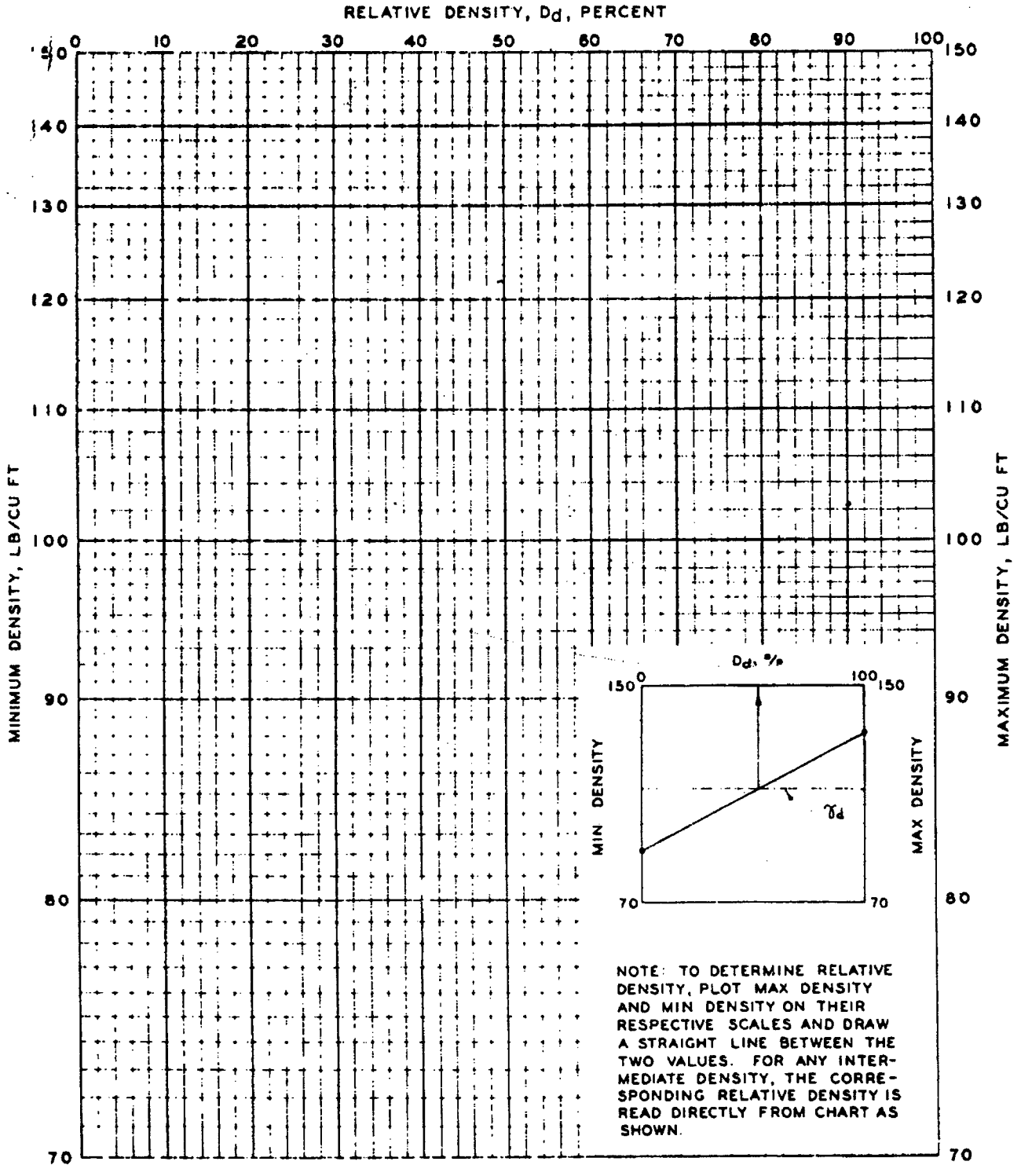


Figure 5. Graphical determination of relative density

(3) Loss of material from mold before weighing. To prevent spilling any material before the mold and contents can be weighed, rap the side of the mold carefully to settle the contents.

c. Maximum Density Determination. (1) Insufficient amplitude of vibratory table under load. Measurements should be made at least once to verify that the requirements given in paragraph 2f are satisfied.

(2) (Rescinded)

(3) Loss of fine material from mold during vibration. Fine particles may escape from the mold during vibration as dust.

(4) Misalignment of guide sleeve with mold. The adjustment of the clamping bolts must be checked periodically to ascertain that misalignment of the guide sleeve will not cause binding of the surcharge weight.

RELATIVE DENSITY

MINIMUM AND MAXIMUM DENSITY DETERMINATIONS

DATE_____

PROJECT_____

BORING NO._____ SAMPLE NO._____

CLASSIFICATION

MOLD NO.	MOLD DIAMETER, IN. =	WEIGHT OF MOLD, W_m, LB =
VOLUME OF MOLD, V_m, CU FT =		END AREA OF MOLD, A_m, SQ FT =

MINIMUM DENSITY

TRIAL NO.			1	2	3	4
WEIGHT LB	MOLD (OR TARE) AND SOIL, DRY	w				
	MOLD (OR TARE)	w_m				
	SOIL, DRY	w_s				
MIN DRY DENSITY, LB/CU FT = w_s/V		γ_d				
MIN DRY DENSITY, AVERAGE			LB/CU FT			

MAXIMUM DENSITY

METHOD USED

TRIAL NO:			1	2	3	4
HEIGHT, IN.	LEFT DIAL READING	h_L				
	RIGHT DIAL READING	h_R				
	AVERAGE DIAL = $(h_L + h_R)/2$	h_{avg}				
	INITIAL DIAL READING	h_o				
	HEIGHT CHANGE = $h_o - h_{avg}$	Δh				
VOL CU FT	VOLUME CHANGE = $(\Delta h/12) \times A_m$	Δv				
	VOLUME OF SOIL = $V_m - \Delta v$	v				
WEIGHT LB	MOLD (OR TARE) AND SOIL, DRY	w				
	MOLD (OR TARE)	w_m				
	SOIL, DRY	w_s				
MAX DRY DENSITY, LB/CU FT = w_s/V		γ_d				
MAX DRY DENSITY, AVERAGE			LB/CU FT			

REMARKS_____

TECHNICIAN_____ COMPUTED BY_____ CHECKED BY_____

ENG FORM NO. 3938
REV JUNE 1970

PLATE XII-1

APPENDIX XIIA:

MODIFIED PROVIDENCE VIBRATED DENSITY TEST

1. INTRODUCTION. The modified Providence vibrated density test is used to determine the maximum density of a cohesionless soil when the use of a vibratory table (as described in Appendix XII, RELATIVE DENSITY) is not feasible, especially in field laboratories. The apparatus and procedure are similar to those used for the Providence vibrated density test,† except that the surcharge is applied with a 38.5-lb weight (equivalent to 1 psi) instead of the 1000-lb calibrated spring (equivalent to 26 psi). In this test, a sample of oven-dried soil is placed in a heavy steel mold, compressed under the surcharge, and vibrated to maximum density by repeatedly striking the side of the mold with a hammer.

2. APPARATUS. The apparatus shall consist of the following:

a. Cylindrical steel mold having a capacity of about 0.16 cu ft and an inside diameter of 7 in., as shown in Figure 1.

b. Surcharge assembly, as shown in Figure 2, consisting of a surcharge weight with guide rod, a surcharge baseplate with lifting rod, and a guide yoke with clamping bolts.

c. Depth gage consisting of a dial indicator, having 0.004 in. graduations and a 2-in. range, fitted with a depth gage base and an extra long stem to reach from the top of the guide yoke to the top of the surcharge baseplate.

d. Calibration bar, metal, 2 in. by 1/8 in. by 7-1/2 in. long.

e. Hammer, 2-1/2-lb ball peen.

f. Pouring device (as shown in Figure 3 of Appendix XII,

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† U. S. Army Engineer Division, New England, CE, New England Division Laboratory Procedure for Performing the Providence Vibrated Density Test to Determine the Maximum Density of Cohesionle ss Soils (Waltham, Mass., August 1957).

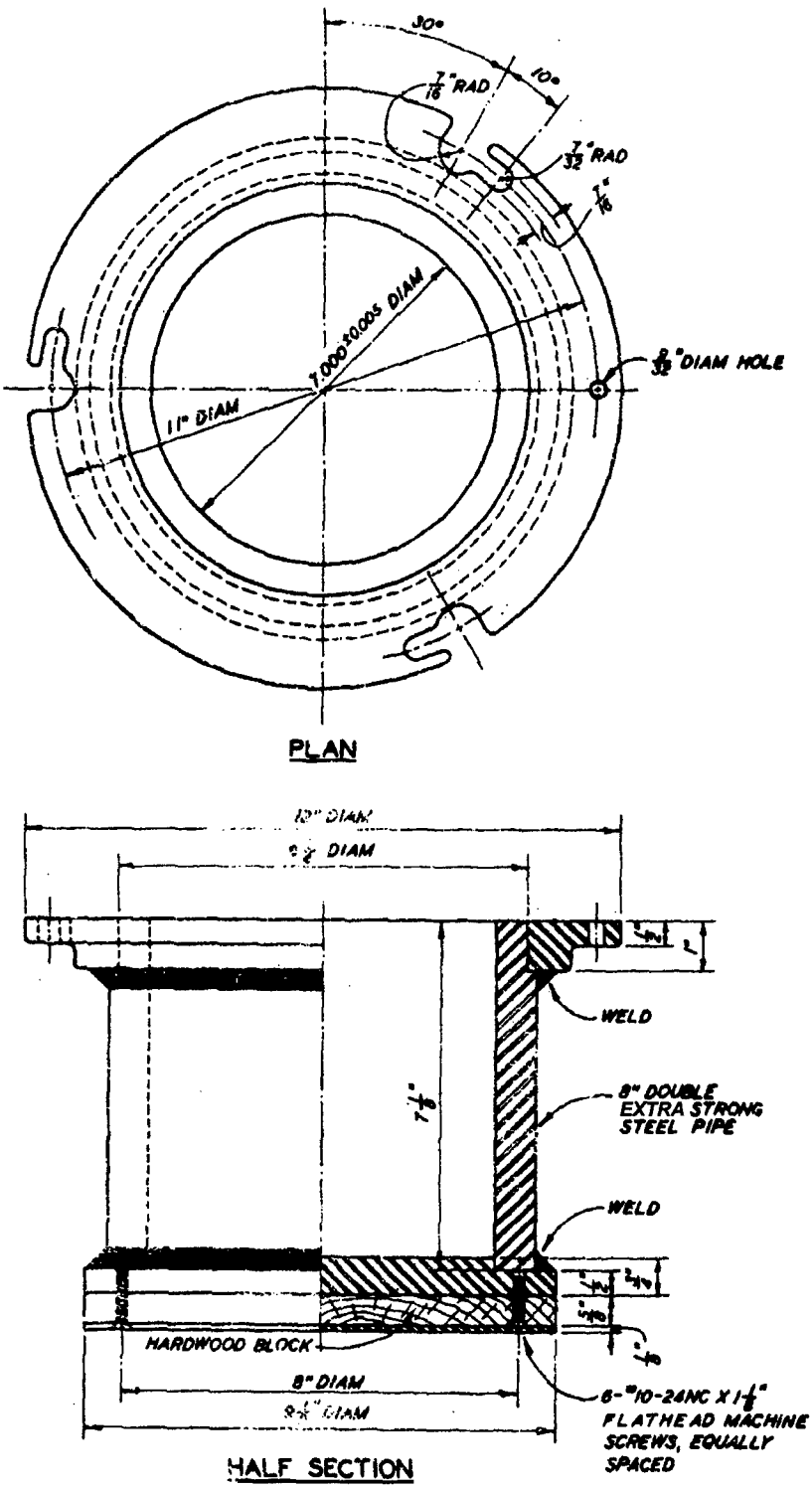


Figure 1. Details of mold

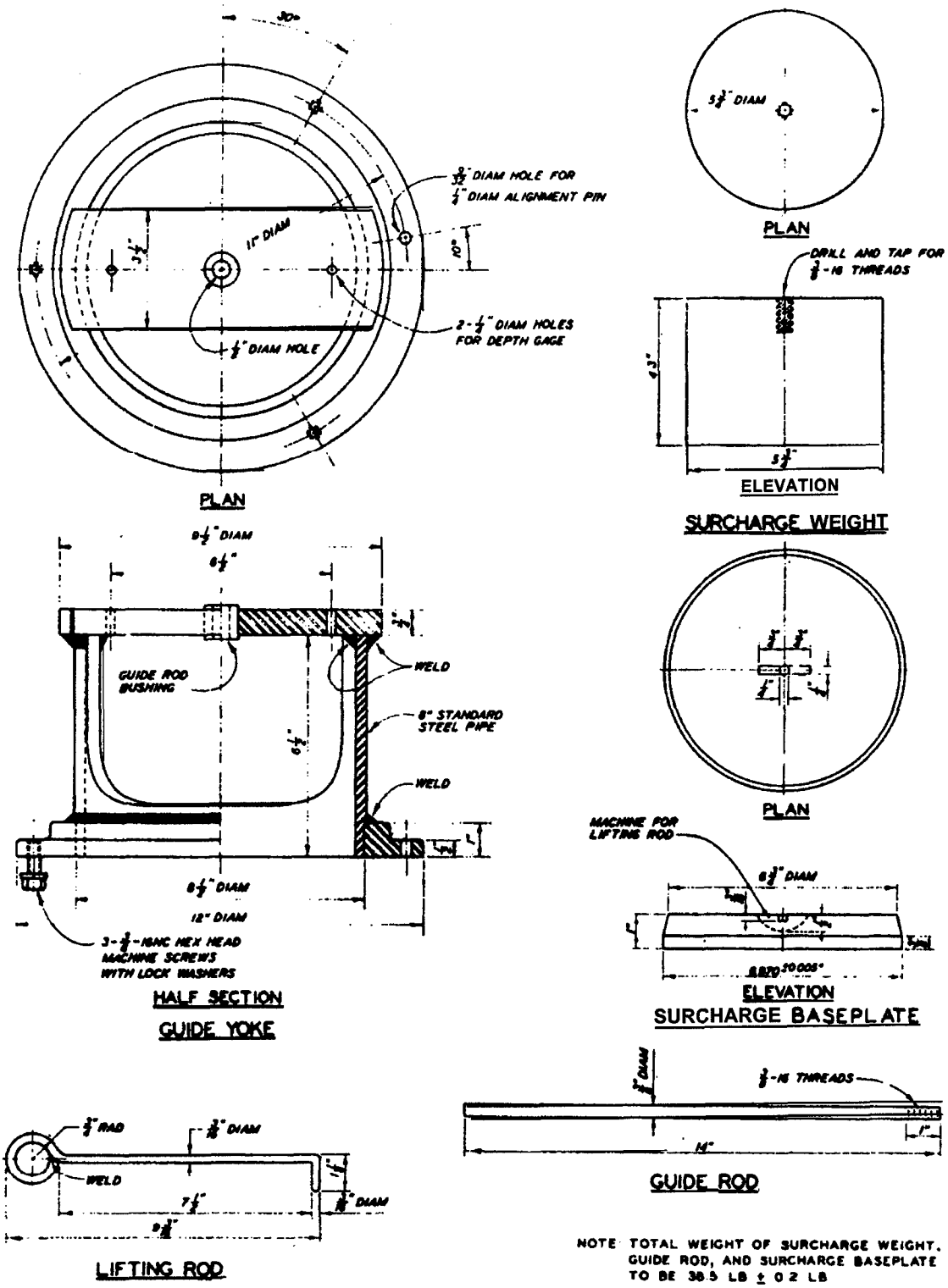


Figure 2. Equipment for maximum density determination

RELATIVE DENSITY) having metal funnels with 1/2- and 1-in.-diameter cylindrical spouts, each attachable to a metal can 6 in. in diameter by 12 in. high.

g. Hand scoop large, metal.

h. Mixing pans, large, metal.

i. Sample splitter or riffle.

j. Straightedge, 15-in., steel.

k. Balance or scale of at least 25-lb capacity sensitive to 0.01 lb.

l. Oven (see Appendix I, WATER CONTENT - GENERAL).

3. **CALIBRATION OF EQUIPMENT.** The mold shall be calibrated as follows:

a. Determine the capacity, or total inside volume, V_m, of the mold to the nearest 0.0001 cu ft.†

b. Determine the inside cross-section area, A_m, at the open end of the mold to the nearest 0.0001 sq ft.

c. Determine the initial dial reading, h_o, in the following manner:

(1) Measure the thickness of the calibration bar, t_c, and of the surcharge baseplate, t_s, to the nearest 0.001 in.

(2) Attach the guide yoke to the mold, tightening the clamping bolts firmly.

(3) Place the calibration bar inside the guide yoke so that it lies on top of the mold in alignment with the top of the yoke.

(4) Place the depth gage on the top of the yoke with the dial indicator stem passing through one of the depth gage holes to rest on top of the calibration bar, and note the dial reading. Then insert the stem through the other hole and note the dial reading.

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† Since the weight, W_m, of this heavy mold, if included as a tare, would detract from the accuracy of weighing the soil in the mold, the soil should be removed from the mold for weighing.

(5) Compute the average, h_r, of the two dial readings.

(6) Compute the initial dial reading, h_o, by the equation

$$h_o = h_r + t_c - t_s$$

4. PREPARATION OF SAMPLE. The sample shall be prepared as described in paragraph 4 of Appendix XII, RELATIVE DENSITY, except that the maximum particle size of the sample must be less than 2-1/2 in. The sample should weigh at least 25 lb.

5. PROCEDURE. a. Minimum Density Determination. The procedure for determining the minimum density shall consist of the following steps:

(1) Proceed in accordance with paragraphs 5a(1) through 5a(3) of Appendix XII, RELATIVE DENSITY.

(2) If the maximum density of the oven-dried sample is not to be determined,† empty the contents of the mold into a mixing pan and weigh the pan and soil to the nearest 0.01 lb. Record the weight on the data sheet.

(3) Paragraph 5a(3) of Appendix XII, RELATIVE DENSITY, and step (2) above should be repeated until consistent results (within 1 percent) are attained.

b. Maximum Density Determination. The procedure for determining the maximum density shall consist of the following steps:

(1) Proceed in accordance with paragraphs 5a(1) through 5a(3) of Appendix XII, RELATIVE DENSITY.

(2) Rap the side of the mold gently with the hammer until the surface of the soil settles at least 1/8 in. below the top of the mold, taking care that the surface of the soil remains parallel with the top of the mold.

(3) Lower the surcharge baseplate onto the surface of the soil

† If the maximum density of the oven-dried sample is to be determined also, proceed in accordance with paragraphs 5b(2) through 5b(8).

and remove the lifting rod. Place the surcharge weight on the surcharge baseplate so that it is centered on the plate. Lower the guide yoke onto the top of the mold, threading the guide rod through the center hole in the top of the yoke, and clamp the yoke firmly to the mold.

(4) Strike the side of the mold between the flanges with sharp, forceful blows of the ball-peen hammer, rotating the mold as each blow is delivered. The blows shall be uniformly distributed over the height and circumference of the mold at a rate not exceeding 40 blows per minute

(5) When the vertical movement of the guide rod becomes imperceptible, place the depth gage on the top of the yoke with the dial indicator stem passing through one of the depth gage holes to rest on top of the surcharge baseplate, and note the dial reading. Then continue striking the mold in cycles of about 25 blows, reinserting the depth gage into the same hole after each cycle and noting the dial reading. The sample shall be considered to have attained the maximum density when the change in dial reading for any 25-blow cycle is less than 0.005 in.

(6) Obtain final dial readings with the depth gage inserted through each of the holes in the yoke. Record the dial readings on the data sheet.

(7) Remove the guide yoke, surcharge weight, and surcharge baseplate from the mold.

(8) Empty the contents of the mold into a mixing pan and weigh the pan and soil to the nearest 0.01 lb. Record the weight on the data sheet.

6. COMPUTATIONS. The computations shall be made as described in paragraph 6 of Appendix XII, RELATIVE DENSITY.

7. POSSIBLE ERRORS. Besides the errors given in paragraphs 7a and 7b of Appendix XII, RELATIVE DENSITY, following are possible errors that would cause inaccurate determinations of maximum density:

a. Loosening of yoke during vibration. The clamping bolts should be checked before each dial reading to ascertain that the yoke is

tightly secured to the mold.

b. Nonuniform distribution of hammer blows over the height and circumference of mold.

c. Vibration of mold stopped too soon.

d. Mold used as tare when determining weight of contents.

APPENDIX XIII:

PINHOLE EROSION TEST FOR IDENTIFICATION OF DISPERSIVE CLAYS

1. CHARACTERISTICS OF DISPERSIVE CLAYS. a. Nature of Dispersive Clays. Dispersive clays are a particular type of soil in which the clay fraction erodes in the presence of water by a process of deflocculation. This occurs when the interparticle forces of repulsion exceed those of attraction so that clay particles are detached and go into suspension. If the water is flowing, as in a crack in an earth dam, the detached clay particles are carried away and piping occurs. Problems involving dispersive clays include piping failure of earth dams, rainfall erosion on slopes of earth dams, erosion of shoreline on reservoirs due to wave action, and erosion of channels (both unlined and lined) constructed in dispersive clays.

b. Influence of Clay-Sized Particles† and Plasticity. Soils with the fraction finer than 0.005 mm $\leq$ 12 percent and with a plasticity index $\leq$ 4 generally do not contain sufficient colloids to support dispersive erosion.††,‡ However, such soils are known to have low resistance to erosion,‡‡ and the dispersion characteristics would add little to the

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† 0.005 mm is American Society for Testing and Materials (ASTM) designation for clay-sized particles (see ASTM D-422-63).

†† J. L. Sherard, L. P. Dunnigan, and R. S. Decker, "Identification and Nature of Dispersive Soils," Journal of the Geotechnical Engineering Division, Vol 102, No. GT4, Apr 1976, pp 287-301.

‡ N. L. Ryker, "Encountering Dispersive Clays on SCS Projects in Oklahoma," Dispersive Clays, Related Piping, and Erosion in Geotechnical Projects, ASTM Special Technical Publication No. 623, American Society for Testing and Materials, May 1977, pp 370-389.

‡‡ H. J. Gibbs, "A Study of Erosion and Tractive Force Characteristics in Relation to Soil Mechanics Properties," Soils Engineering Report No. EM-643, 23 Feb 1962, U. S. Department of the Interior, Bureau of Reclamation, Denver, Colo.

known field performance of the soils.

2. IDENTIFICATION OF DISPERSIVE CLAYS. a. Introduction. Identification of dispersive clays may be required for earth structures not yet constructed, for existing earth structures, and for natural soil deposits. Positive identification of dispersive clays is by observed performance of the soil in the field.* Dispersive clays cannot be identified by conventional index tests such as particle size distribution, Atterberg limits, and compaction characteristics.

b. Laboratory Tests Used to Identify Dispersive Clays. Four laboratory tests commonly used to identify dispersive clays are the Crumb test, Soil Conservation Service (SCS) dispersion test, soil pore water chemistry correlation, and the pinhole erosion test.††

(1) The Crumb test (procedure given in Plate XIII-1) is often used as an adjunct to other tests for identifying dispersive clays. However, the Crumb test is a useful indicator only in one direction. If the Crumb test indicates dispersion (Crumb reading 3 or 4), the soil is probably dispersive; however many dispersive soils, particularly kaolinitic soils, do not react to the Crumb test (i.e., give Crumb readings of I or 2).‡

(2) The SCS dispersion test has been used to identify

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* P. Forsythe, "Experience in Identification and Treatment of Dispersive Clays in Mississippi Dams," Dispersive Clays, Related Piping, and Erosion in Geotechnical Projects, ASTM Special Technical Publication No. 623, American Society for Testing and Materials, May 1977, pp 135-155.

†† J. L. Sherard et al., " Pinhole Test for Identifying Dispersive Soils ," Journal of the Geotechnical Engineering Division, Vol 102, No. GT1, Jan 1976, pp 69-85.

Sherard, Dunnigan, and Decker, op. cit.

‡ G. G. S. Holmgren, and C. P. Flanagan, "Factors Affecting Spontaneous Dispersion of Soil Materials as Evidenced by the Crumb Test," Dispersive Clays, Related Piping, and Erosion in Geotechnical Projects, ASTM Special Technical Publication No. 623, American Society for Testing and Materials, May 1977, pp 218-239.

dispersive clays. Available results indicate for soils with SCS dispersion < 35 percent, dispersive erosion will not be a problem; for soils with SCS dispersion from 35 to 50 percent, dispersive erosion may or may not occur; and for soils with SCS dispersion > 50 percent, dispersive erosion will be a problem.†,††,‡ The SCS dispersion test has about 85 percent reliance in predicting dispersive performance (about 85 percent of dispersive soils show more than 35 percent SCS dispersion).††

(3) Sherard et al.,‡‡ have obtained a relationship between dispersion and soil pore water chemistry based on pinhole erosion tests and observed dispersion erosion in nature, as shown in Figure 1. The soil pore water correlation has about 85 percent reliance in predicting dispersive performance.††

(4) The pinhole erosion test is the most reliable test for identifying dispersive soils.†,‡‡ Figure 2 shows a schematic representation of the pinhole erosion test. In conducting the test, distilled water under a low hydraulic head is caused to flow through a small diameter hole in the soil specimen. For dispersive soils, the flow emerging from the soil specimen is cloudy and the hole rapidly enlarges. For nondispersive soils, the flow is clear and the hole does not enlarge. The pinhole erosion test was developed for the purpose of identifying dispersive soils and is not intended to be a geometrically scaled model of an actual prototype structure. Since the theory of similitude was not used in the design of the test, quantitative data

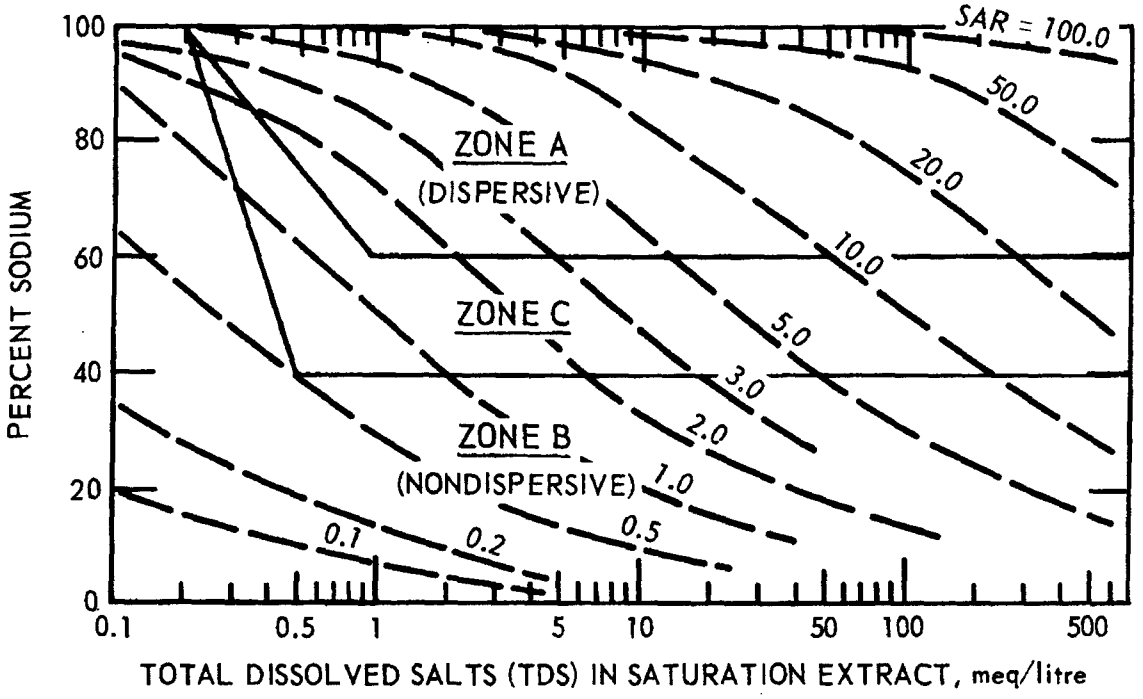
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† Sherard, Dunnigan, and Decker, op. cit.

†† R. S. Decker and L. P. Dunnigan, "Development and Use of the SCS Dispersion Test," Dispersive Clays, Related Piping, and Erosion in Geotechnical Engineering Projects, ASTM Special Technical Publication No. 623, American Society for Testing and Materials, May 1977, pp 94-109.

‡ N. L. Ryker, op. cit.

‡‡ J. L. Sherard et al., op. cit.



NOTE: RELATIONSHIP SHOWN IS VALID ONLY WHEN ERODING WATER IS RELATIVELY PURE.

$$\text{PERCENT SODIUM} = \frac{Na\ (100)}{TDS} = \frac{Na\ (100)}{Ca + Mg + Na + K}$$

$$TDS = Ca + Mg + Na + K$$

$$SAR = \frac{Na}{\sqrt{0.5\ (Ca + Mg)}}, \text{ ALL IN meq/litre}$$

Figure 1. Relationship between dispersion and soil pore water chemistry based on pinhole erosion tests and experience with erosion in nature (after Sherard et al.)

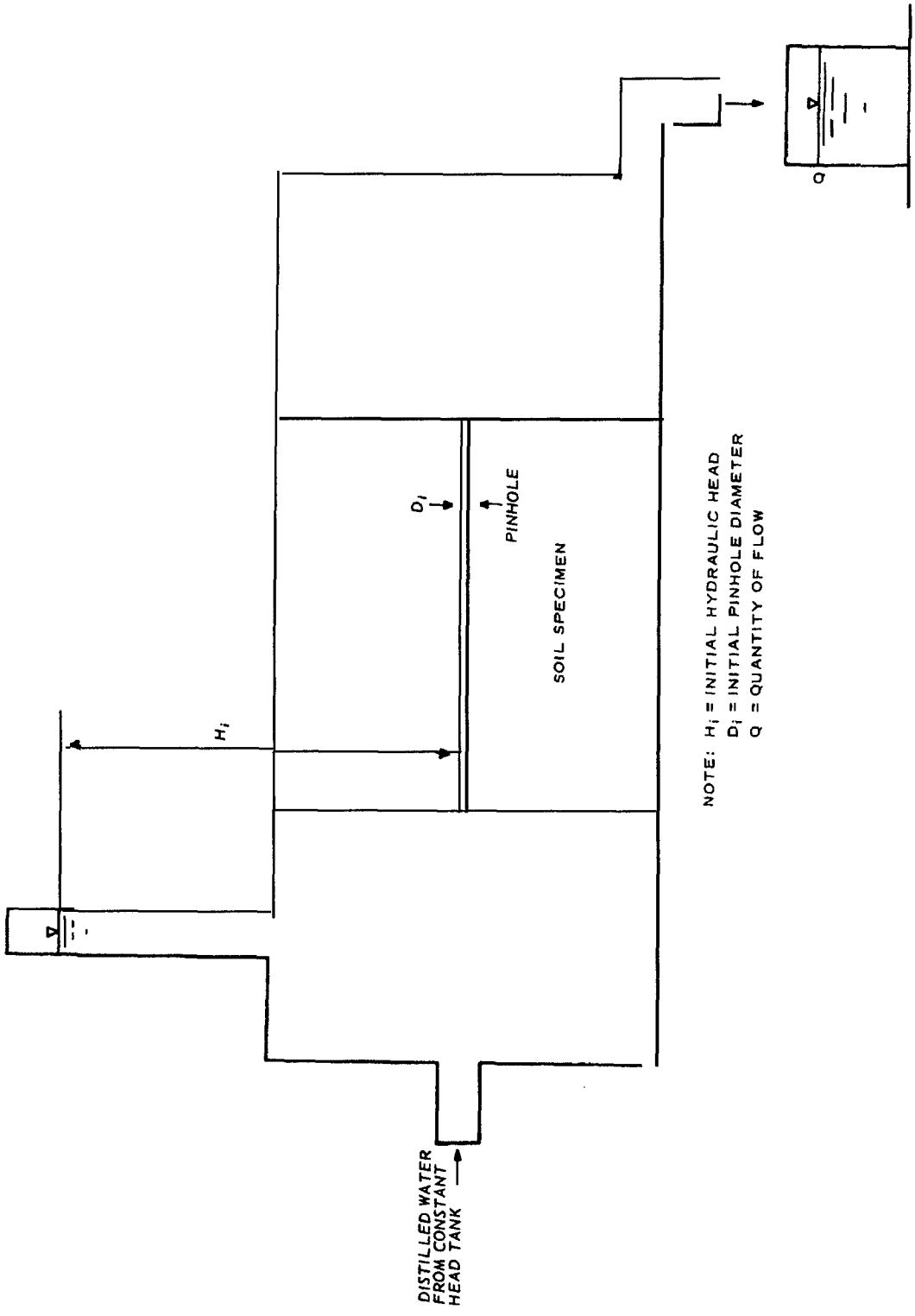


Figure 2. Schematic representation of WES pinhole erosion apparatus

are not obtained on the quantity of flow through the pinhole, amount of soil erosion, or rate of soil erosion. Two limitations of the pinhole erosion test for identifying dispersive soils have been observed. Undistrubed soil samples of high sensitivity (ratio of the peak undrained strength of the soil in a natural state to the peak undrained strength after it has been remolded without change in water content) may be classified as dispersive from the pinhole erosion test, while in nature the soil may be resistant to erosion.† Apparently, the natural structure of the soil is destroyed by punching the pinhole in the undisturbed soil specimen and a reaction analogous to dispersion is obtained in the pinhole erosion test. Soils with high sodium (>80 percent) and low total dissolved solids (< 0.4 meq/ℓ) in the soil pore water may show nondispersive in the pinhole erosion test, while the soil may exhibit dispersive performance in the field.†† This may occur because a decrease in the concentration gradient between the soil pore water and eroding fluid (distilled water ≃ 0.0 meq/ℓ for pinhole erosion test) results in a decrease in the erosion rate for soils.‡ However, available data from case histories indicate very few soils with total dissolved solids <1.0 meq/ℓ for which dispersive performance has been observed in the field.‡‡

c. Field Tests Used to Identify Dispersive Clays. Four field tests that have been used to identify dispersive clays are the Crumb test (also used as a laboratory test), the ultraviolet light, the modified

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† 0. Dascal, G. Pouliot, and J. Hurtubise, "Erodibility Tests on a Sensitive, Cemented Marine Clay (Champlain Clay)," Dispersive Clays, Related Piping, and Erosion in Geotechnical Projects, ASTM Special Technical Publication No. 623, American Society for Testing and Materials, May 1977, pp 74-93.

†† Forsythe, op. cit.

‡ A. Sargunam, "Influence of Mineralogy, Pore Fluid Composition and Structure on the Erosion of Cohesive Soils," Ph.D. Dissertation, June 1973, University of California, Davis, Calif.

‡‡ Sherard, Dunnigan, and Decker, op. cit.

hydrometer or Dilution-Turbidity test, and determination of soil pore water chemistry by sodium electrode and chemical reagents or Wheatstone bridge.

(1) The Crumb test, which may be used in the laboratory or field, has been described previously.

(2) The ultraviolet light has been used to indicate the presence of sodium in the soil.* Uranyl acid is mixed with the soil and the intensity and amount of fluorescence under ultraviolet light is observed The ultraviolet light has shown about 40 percent reliance in predicting dispersive performance of soils in Mississippi.

(3) The modified hydrometer or Dilution-Turbidity test has been used to identify dispersive clays.†,†† Prior to using the modified hydrometer test to identify dispersive clays in a particular area, the test results must be correlated with laboratory tests to establish a range of values. For example, test data from the Bluff Hills region of Mississippi indicate for a turbidity ratio < 4, dispersive erosion will be a problem; for a turbidity ratio 4-9, dispersive erosion may or may not occur; and for a turbidity ratio > 9, dispersive erosion will not be a problem.*

(4) Two methods have been developed for determination of soil pore water chemistry in the field to use with the correlation shown in Figure 1 and to identify dispersive clays.‡ The first method involves a sodium electrode and chemical reagents to determine the percent sodium and total dissolved cations (calcium, magnesium, and

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* Forsythe, op. cit.

†† Ryker, op. cit.

‡C. P. Flanagan and G. G. S. Holmgren, "Field Methods for Determination of Soluble Salts and -Percent Sodium From Extract for Identifying Dispersive Clay Soils," Dispersive Clays, Related Piping, and Erosion on Geotechnical Projects, ASTM Special Technical Publication No. 623, American Society for Testing and Materials, May 1977, pp 121-134.

sodium). This method does not determine potassium, which exists in small quantities ($\leq$ 1.0 meq/ℓ) in most soils. The second method uses a Wheatstone bridge to determine the percent sodium and total dissolved cations (calcium, magnesium, sodium, and potassium).

d. Recommended Procedure for Identification of Dispersive Soils. Clays with the fraction finer than 0.005 mm $\leq$ 12 percent and with a plasticity index $\leq$ 4 generally do not contain sufficient colloids to support dispersive erosion. The pinhole erosion test is the recommended laboratory test for identifying dispersive clays. A Crumb test (procedure given in Plate XIII-I) should be run on each soil tested in the pinhole erosion apparatus. There is no generally reliable field test for identifying dispersive clays.

(1) To identify dispersive clays for earth structures not yet constructed, pinhole erosion tests will be conducted on compacted soil samples taken from proposed borrow areas.

(2) To identify dispersive clays in existing earth structures or natural soil deposits, pinhole erosion tests will be conducted on undisturbed soil samples.

3. WES PINHOLE EROSION APPARATUS. Detailed drawings for the U. S. Army Engineer Waterways Experiment Station (WES) pinhole erosion apparatus are given in Plates XIII-2 to XIII-4. The apparatus, shown schematically in Figure 2, is a simplified version of a laboratory erosion test apparatus constructed previously at WES.† The apparatus is designed to accommodate three different size specimens: (a) compacted specimens 1.312 in. in diameter by 2.816 in. high (Harvard miniature compaction mold), (b) compacted specimens 4.00 in. in diameter by 4.59 in. high (standard compaction mold), and (c) undisturbed

† E. B. Perry, "Piping in Earth Dams Constructed of Dispersive Clay; Literature Review and Design of Laboratory Tests," Technical Report S-75-15, Nov 1975, U. S. Army Engineer Waterways Experiment Station, CE, Vicksburg, Miss.

specimens encased in Shelby-tube 3.125 in. in outside diameter by 4.60 in. high. Accessory equipment needed includes: (a) de-aired distilled water; (b) graduated cylinders of 10-, 25-, 50-, 100-, and 250-ml capacity; (c) stopwatch reading to 0.1 sec; (d) centigrade thermometer with range 0 to 50C and accurate to 0.1C; (e) modeling clay,† plus 1/8- and 1/4-in. circular hardware cloths cut to fit inside the specimen tube; (f) pea gravel, 1/4- to 3/8-in. size; and (g) 1/16-in.-diameter steel welding rod.

4. CALIBRATION OF PINHOLE EROSION APPARATUS. a. Concept. To interpret the results of the pinhole erosion test and to develop a classification system containing intermediate grades between dispersive and nondispersive clays, it is necessary to determine the relationship between quantity of flow and initial hydraulic head as a function of the size of the pinhole, as shown in Figure 3. This relationship is determined by substituting aluminum cylinders with varying pinhole diameter (Plate XIII-4) for the soil specimen and measuring the quantity of flow for various hydraulic heads. Two calibrations are required: (1) 4.60-in.-long aluminum cylinders with varying pinhole diameter, and (2) 2.82-in.-long aluminum cylinders with varying pinhole diameters. These calibrations should be conducted for each pinhole erosion apparatus used.

b. Assembly of Apparatus. To calibrate the pinhole erosion apparatus, the temperature of the distilled water is recorded both before and after the test. The 4.60-in.-long aluminum cylinder with the 1/16-in.-diameter pinhole is pushed into the specimen tube and positioned longitudinally in the center of the specimen tube as shown (for a soil specimen) in Plate XIII-2. The 1/4-in. hardware cloth is placed next to the exit end of the aluminum cylinder (away from the distilled water entrance) and positioned so that the pinhole is centered in an opening in the hardware cloth. The 1/8-in. hardware cloth is

† Clayola modeling clay, Binney and Smith, Inc., 380 Madison Avenue, New York, N. Y. 10017.

*

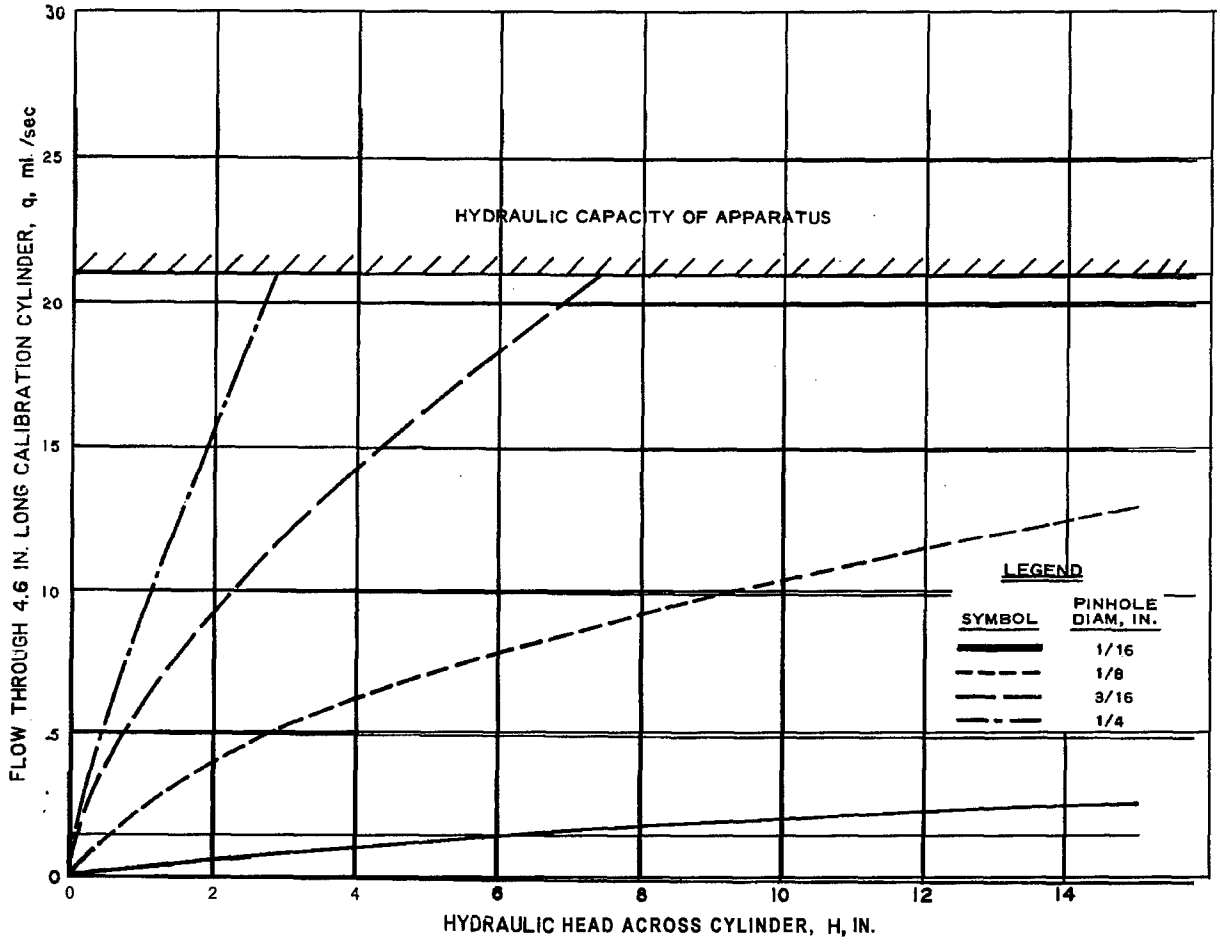


Figure 3. Typical calibration curve for pinhole apparatus using 4.6-in.-long specimen

placed next to the 1/4-in. hardware cloth. Modeling clay is placed around the perimeter of the hardware cloths to hold them in place. Pea gravel (1/4- to 3/8-in. size) is placed next to the hardware cloths, and the drain plate (Plate XIII-3) is attached to the exit end of the specimen tube. The 1/4-in. hardware cloth is placed next to the entrance end of the aluminum cylinder and pusitioned so that the pinhole is centered in an opening in the hardware cloth. The 1/8-in. hardware cloth is placed next to the 1/4-in. hardware cloth. Modeling clay is placed around the perimeter of the hardware cloths to hold them in place and to provide a watertight seal between the beveled edge of the aluminum cylinder

and the inside of the specimen tube. Pea gravel (1/4- to 3/8-in. size) is placed next to the hardware cloths and the pressure plate (Plate XIII-31 is attached to the entrance end of the specimen tube, completing the assembly of the pinhole erosion test apparatus.

c. Test Procedure for Calibration. Distilled water from the constant head tank is caused to flow through the pinhole in the aluminum cylinder under a hydraulic head of 1 in. for a period of 2 to 4 min (until the quantity of flow becomes constant). The quantity of flow is measured continuously with a stop watch (nearest 0.1 sec) and graduated cylinders (10, 25, 50, 100, or 250 ml) and recorded on the data sheet (Plate XIII-5). The quantity of flow is measured for hydraulic heads of 1, 2, 3, 5, 7, 9, 11, 13, and 15 in. This process is then repeated using the aluminum cylinders with 1/8-, 3/16-, and 1/4-in.-diameter pinholes. The hydraulic capacity (maximum quantity of water which will flow through the pinhole erosion apparatus for a given hydraulic head and pinhole diameter) of the pinhole erosion apparatus may be reached with the 1/4-in.-diameter pinhole at less than 15 in. of hydraulic head. This completes the calibration for the 4.60-in.-long aluminum cylinders. A similar calibration will then be conducted using the 2.816-in.-long aluminum cylinders with 1/16-, 1/8-, 3/16-, and 1/4-in.-diameter pinholes.

d. Presentation of Calibration Results. The results of the calibration test for the 4.6-in.-long aluminum cylinders are plotted in a manner similar to that shown in Figure 3. The measured quantities of flow from the calibration test are used to prepare a classification of test results for the pinhole erosion test using 4.6-in.-long specimens (see Fig. 4). The classification shown in Figure 4 is depicted graphically in Figure 5.

5. PREPARATION OF COMPACTED SPECIMENS FOR PINHOLE EROSION TESTING. To identify dispersive clays for earth structures not yet constructed, pinhole erosion tests will be conducted on

*

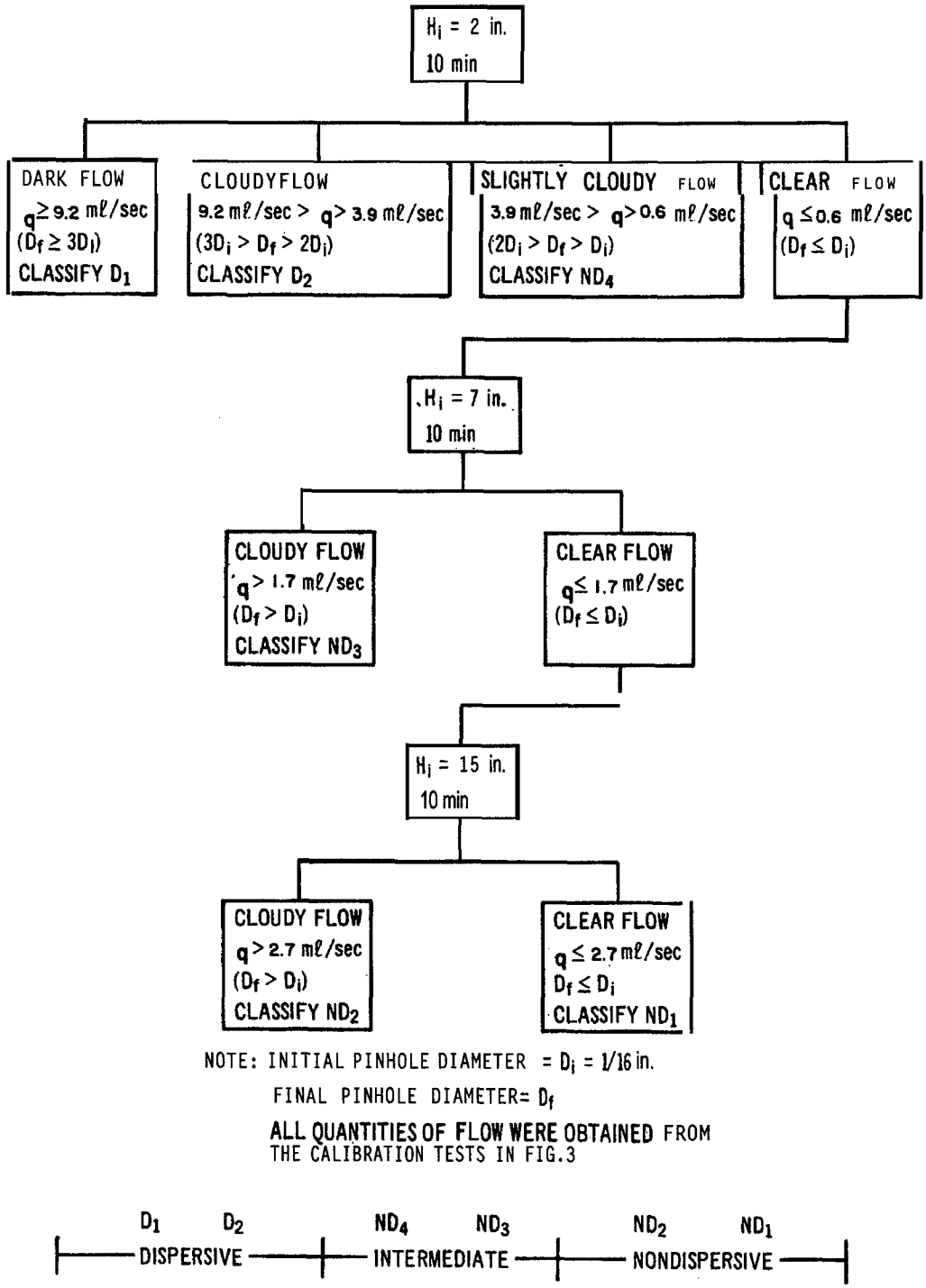


Figure 4. Typical sequence of testing and classification of test results for pinhole erosion test using 4.6-in.-long specimens *

*

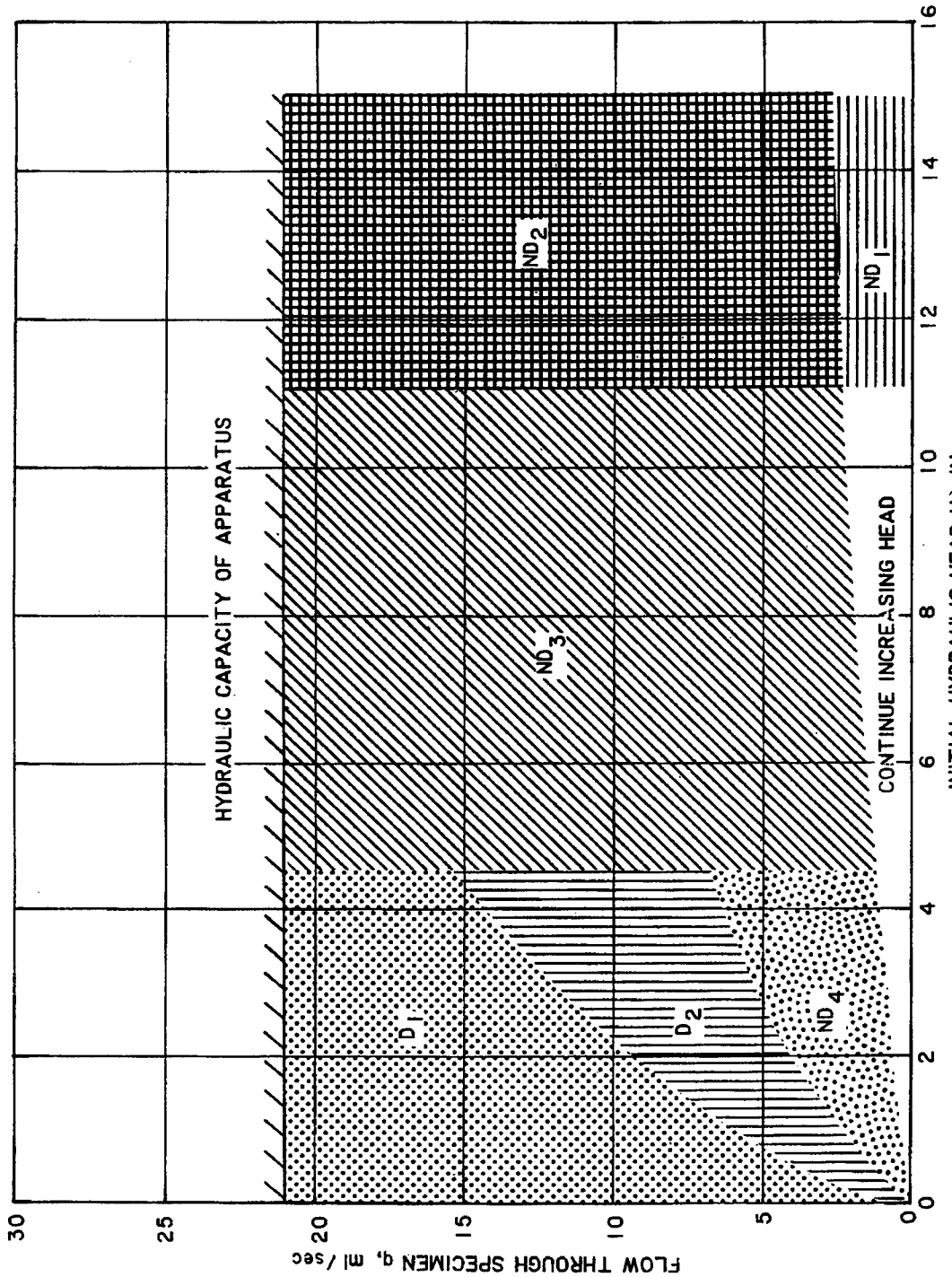


Figure 5. Typical graphical illustration of classification of pinhole erosion test results using 4.6-in.-long specimens

*

compacted specimens from soil samples taken from proposed borrow areas. The soil sample shall be maintained at natural water content prior to testing.† If the soil sample contains gravel-size particles, they will be removed by pushing the material through the No. 4 sieve (0.187-in. Opening). The soil specimen tested in the pinhole erosion apparatus shall be compacted at the same water content and dry density as those proposed for the earth structure in the field. If the natural water content is greater than that proposed for construction, the soil shall be air-dried to the construction water content. If the natural water content is less than the proposed construction water content, distilled water shall be added to bring the soil to the construction water content. The type of compaction (kneading, dynamic, or other) used in the laboratory to prepare the soil specimens for pinhole erosion testing shall simulate the field compaction. Where the soil is to be compacted in the field with a sheepsfoot roller, the soil specimens for pinhole erosion testing shall be compacted with a laboratory kneading compaction apparatus (see Appendix X, TRIAXIAL COMPRESSION TESTS, pp X-14 and X-15).†† When the soil is to be compacted in the field with hand-operated tampers, such as in the vicinity of an outlet conduit, the soil specimens for pinhole erosion testing shall be compacted with a laboratory dynamic (impact) compaction apparatus (see Appendix VI, COMPACTION TESTS, p VI-13). Following compaction, the soil specimen for pinhole erosion testing shall be tested immediately (the soil specimens shall not be allowed to cure following compaction) in the pinhole erosion apparatus.

6. PREPARATION OF UNDISTURBED SPECIMENS FOR PINHOLE

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† Sherard, et al., op. cit.

†† S. D. Wilson, "Suggested Method of Test for Moisture-Density Relations of Soils Using Harvard Compaction Apparatus," Proceedings for Testing Soils, American Society for Testing and Materials, 4th Edition, Dec 1964, pp 160-162.

EROSION TESTING. To identify dispersive clays in natural deposits or existing compacted earth structures, the pinhole erosion test will be conducted on undisturbed soil samples. The soil sample is not removed from the Shelby tube. A 4.6-in.-long section is cut from the Shelby tube using a horizontal band saw, and the soil specimen encased in the Shelby tube is placed in the pinhole erosion apparatus for testing. If undisturbed block samples are available, a 4.6-in.-long section of Shelby tube with a sharp cutting edge can be pushed into the block sample to obtain a soil specimen for pinhole erosion testing.

7. PINHOLE EROSION TEST PROCEDURE. a. General. The identifying information for the soil specimen is recorded on the data sheet (Plate XIII-5 is a suggested form). The temperature of the eroding fluid (distilled water) is recorded both before and after the test. The curing time (time lapse between compaction and pinhole erosion testing) is recorded for remolded specimens. Although zero curing time is specified for the pinhole erosion test, some time will be consumed in assembling the apparatus, punching the pinhole in the soil specimen, etc.

b. Assembly of Apparatus. To assemble the pinhole erosion apparatus, the soil specimen is positioned longitudinally in the center of the specimen tube as shown in Plate XIII-2 for a compacted specimen 4.00 in. in diameter by 4.60 in. high. The same procedure is employed when testing an undisturbed soil specimen or a compacted soil specimen 1.3125 in. in diameter by 2.816 in. high (these soil specimens encased in adapters (Plate XIII-3) are positioned longitudinally in the center of the specimen tube). The pinhole punch guide assembly (Plate XIII-4) is attached to the exit end of the specimen tube (away from the distilled water entrance), and a 1/16-in.-diameter steel welding rod is pushed through the pinhole punch guide until the rod just touches (marks) the exit end of the soil specimen. The pinhole punch guide is removed from the exit end of the specimen tube, and

the 1/4-in. hardware cloth is placed next to the exit end of the soil specimen and positioned so that the pinhole mark is centered in an opening in the hardware cloth. The 1/8-in. hardware cloth is placed next to the 1/4-in. hardware cloth, and modeling clay is placed around the perimeters of the hardware cloths to hold them in place. Pea gravel (1/4- to 3/8-in. size) is placed next to the hardware cloths and the drain plate (Plate XIII-3) is attached to the exit end of the specimen tube. The pinhole punch guide assembly is attached to the entrance end of the specimen tube, and a 1/16-in.-diameter steel welding rod is used to punch a pinhole longitudinally through the center of the soil specimen. The pinhole punch guide assembly is removed, and the 1/4-in. hardware cloth is placed next to the entrance end of the soil specimen and positioned so that the pinhole is centered in an opening in the hardware cloth. The 1/8-in. hardware cloth is placed next to the 1/4-in. hardware cloth, and modeling clay is placed around the perimeter of the hardware cloths to hold them in place and to provide a watertight seal between the soil specimen and the inside of the specimen, tube, as shown in Plate XIII- 2. When the soil specimen is encased in an adapter, modeling clay is used to provide a watertight seal between the beveled edge of the adapter (Plate XIII-3) and the inside of the specimen tube. Pea gravel (1/4- to 3/8-in. size) is placed next to the hardware cloths, and the pressure plate (Plate XIII-3) is attached to th.e entrance end of the specimen tube completing the assembly of the pinhole erosion test apparatus.

c. Test Procedure. The 1/4-in. pipe plug, located on top of the specimen tube next to the manometer (Plate XIII-2) is removed, and the entrance end of the specimen tube containing the pea gravel is filled with distilled water. The 1/4-in. pipe plug is placed back into position, and distilled water from the constant head tank is caused to flow through the pinhole in the soil specimen under a hydraulic head of 2 in. for 10 min (if no flow occurs under the 2-in. hydraulic head,

remove the pressure plate, pea gravel, and hardware cloths, attach the pinhole punch guide assembly to the entrance end of the specimen tube, and repunch the pinhole in the soil specimen). The quantity of flow is measured continuously with a stopwatch (nearest 0.1 sec) and graduated cylinders (10, 25, or 50 ml) and recorded on the data sheet (Plate XIII- 5). The color of the water in the flask (clear, cloudy, or dark) is observed and recorded on the data sheet. At the end of the 10-min flow under the 2-in. hydraulic head, the quantity of flow (ml/sec) is computed. If the quantity of flow under 2-in. hydraulic head increases with time showing a cloudy or dark color and after a 10-min flow is greater than the measured quantity of flow obtained when the pinhole erosion apparatus was calibrated using the aluminum cylinder with the 1/16-in.-diameter pinhole, i.e., $D_f > D_i$, the test is concluded. If the quantity of flow under 2-in. hydraulic head does not increase with time and remains clear and after a 10-min flow is approximately equal to or less than the measured quantity of flow obtained when the pinhole erosion apparatus was calibrated using the aluminum cylinder with the 1/16-in.-diameter pinhole, i.e., $D_f \leq D_i$, the hydraulic head is raised to 7 in., and the quantity of flow is measured continuously for 10 min. If the quantity of flow under a 7-in. hydraulic head increases with time showing a cloudy color and after a 10-min flow is greater than the measured quantity of flow obtained when the pinhole erosion apparatus was calibrated using the aluminum cylinder with the 1/16-in.-diameter pinhole, i.e., $D_f > D_i$, the test is concluded. If the quantity of flow under 7-in. hydraulic head does not increase with time and remains clear and after a 10-min flow is approximately equal to or less than the measured quantity of flow obtained when the pinhole erosion apparatus was calibrated using the aluminum cylinder with the 1/16- in. -diameter pinhole, i.e., $D_f \leq D_i$, the hydraulic head is raised to 15 in., and the quantity of flow is measured continuously for 10 min; then the test is concluded.

8. PRESENTATION AND ANALYSIS OF PINHOLE EROSION TEST RESULTS. a. Presentation of Test Results. Plates XIII-6 to XIII-9 present the test results from the pinhole erosion test. These plates are obtained from the calibration of the pinhole erosion apparatus (see Fig. 3). Typical test results for a dispersive clay and a nondispersive clay are also shown in these plates.

b. Analysis of Test Results. The primary differentiation between dispersive and nondispersive clays is given by the pinhole erosion test results under a hydraulic head of 2 in.† For dispersive clays, the quantity of flow under a hydraulic head of 2 in. continuously increases and may reach a maximum value that is limited by the hydraulic capacity of the pinhole erosion apparatus, in less than 10 min. For nondispersive clays, under a hydraulic head of 2 in. the flow will be clear or slightly cloudy. As shown in Figure 4, the pinhole erosion test results are classified as D_1, D_2, ND_4, ND_3, ND_2, or ND_1.† Dispersive clays are classified as D_1 or D_2, intermediate clays as ND_4 or ND_3, and nondispersive clays as ND_2 or ND_1.

9. POSSIBLE ERRORS WITH PINHOLE EROSION TEST. As discussed previously, undisturbed soil specimens of high sensitivity (ratio of the peak undrained strength of the soil in a natural state to the peak undrained strength of the soil after it has been remolded without change in water content), may be classified as dispersive from the pinhole erosion test, while in nature the soil may be resistant to erosion.†† Soils with high sodium (> 80 percent) and low total dissolved solids (< 0.4 meq/ℓ) in the soil pore water may show nondispersive in the pinhole erosion test while the soil may exhibit dispersive performance

† Sherard et al., op. cit.

†† Dascal, Pouliot, and Hurtubise, op. cit.

in the field.† However, available data from case histories indicate very few soils with total dissolved solids <1.0 meq/ℓ for which dispersive performance has been observed in the field.††

† Forsythe, op. cit.

†† Sherard, Dunnigan, and Decker, op. cit.

Crumb Test

1. The Crumb test is run using distilled demineralized water. Small (5-10 g) crumbs of soil at natural water content are carefully placed on the bottom of a 100-ml clear glass beaker filled with distilled demineralized water. The behavior of the soil crumb is observed and readings are taken after 10 and 30 min.

2. Dispersion is detected by the formation of a colloidal cloud, which appears as a fine misty halo around the soil crumb. The Crumb test is rated for reaction or colloidal cloud formation as follows:

1 = no sign of cloudy water caused by colloidal suspension.

2 = bare hint of colloidal cloud formation at surface or soil crumb.

3 = easily recognized colloidal cloud covering one fourth to one half of the bottom of the glass container.

4 = strong reaction with colloidal cloud covering most of the bottom of the glass container.

3. Since the Crumb test involves a small quantity of soil, several tests should be run on each soil sample before making an evaluation. The Crumb test may be used as an indicator of field performance of dispersive soils using the following evaluation of soil crumb reaction:

No dispersion problem = 1
Possible dispersion problem = 2
Definite dispersion problem = 3 or 4

4. The Crumb test is a useful indicator only in one direction. If the Crumb test indicates dispersion (Crumb reading 3 or 4), the soil is probably dispersive; however, many dispersive soils, particularly kaolinitic soils, do not react to the Crumb test (i.e., give Crumb reading of 1 or 2).

PLATE XIII-1

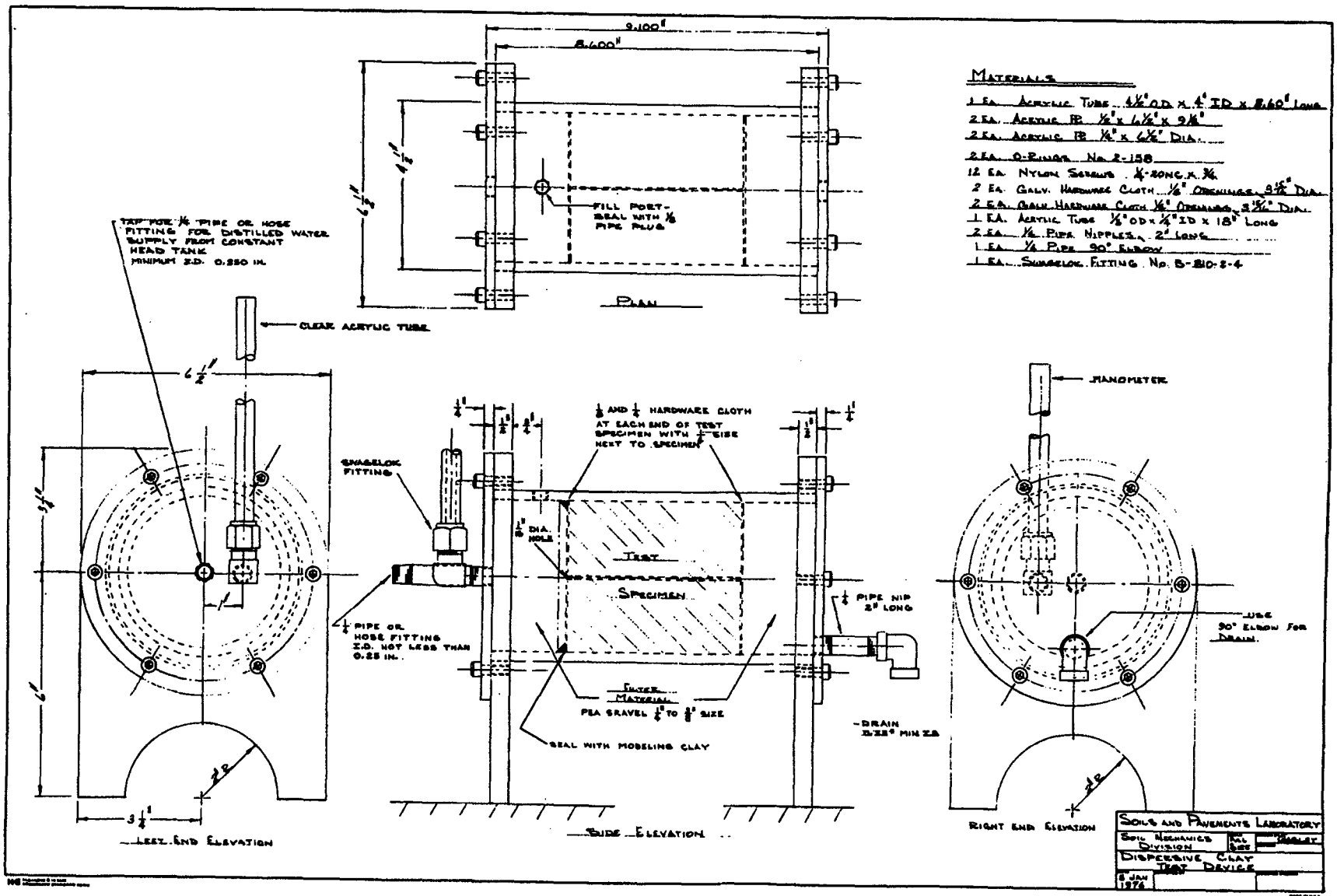

9.100"
8.400"
4½"
6½"
PLAN
FILL PORT - SEAL WITH ⅛ PIPE PLUG
TAP FOR ⅛ PIPE OR HOSE FITTING FOR DISTILLED WATER SUPPLY FROM CONSTANT HEAD TANK MINIMUM I.D. 0.250 IN.
CLEAR ACRYLIC TUBE
6½"
9½"
6"
3¼"
LEFT END ELEVATION
¼ AND ⅛ HARDWARE CLOTH AT EACH END OF TEST SPECIMEN WITH ¼" SIDE NEXT TO SPECIMEN
SWAGELOK FITTING
½" DIA. HOLE
TEST
SPECIMEN
¼ PIPE NIP 2" LONG
¼ PIPE OR HOSE FITTING I.D. NOT LESS THAN 0.25 IN.
FILTER MATERIAL
PEA GRAVEL ¼" TO ⅜" SIZE
SEAL WITH MODELING CLAY
DRAIN 0.25" MIN I.D.
SIDE ELEVATION
MANOMETER
USE 90° ELBOW FOR DRAIN.
RIGHT END ELEVATION
Materials
1 EA. ACRYLIC TUBE 4½" O.D. x 4" I.D. x 8.60" LONG
2 EA. ACRYLIC PL ½" x 6½" x 9½"
2 EA. ACRYLIC PL ½" x 6½" DIA.
2 EA. O-RINGS NO. 2-158
12 EA. NYLON SCREWS ¼-20NC x ¾
2 EA. GALV. HARDWARE CLOTH ⅛" OPENINGS 3½" DIA.
2 EA. GALV. HARDWARE CLOTH ¼" OPENINGS 3½" DIA.
1 EA. ACRYLIC TUBE ½" OD x ¼" ID x 18" LONG
2 EA. ¼ PIPE NIPPLES 2" LONG
1 EA. ¼ PIPE 90° ELBOW
1 EA. SWAGELOK FITTING NO. B-810-2-4
SOILS AND PAVEMENTS LABORATORY
SOIL MECHANICS DIVISION
DISPERSIVE CLAY TEST DEVICE
6 JAN 1976

PLATE XIII-2

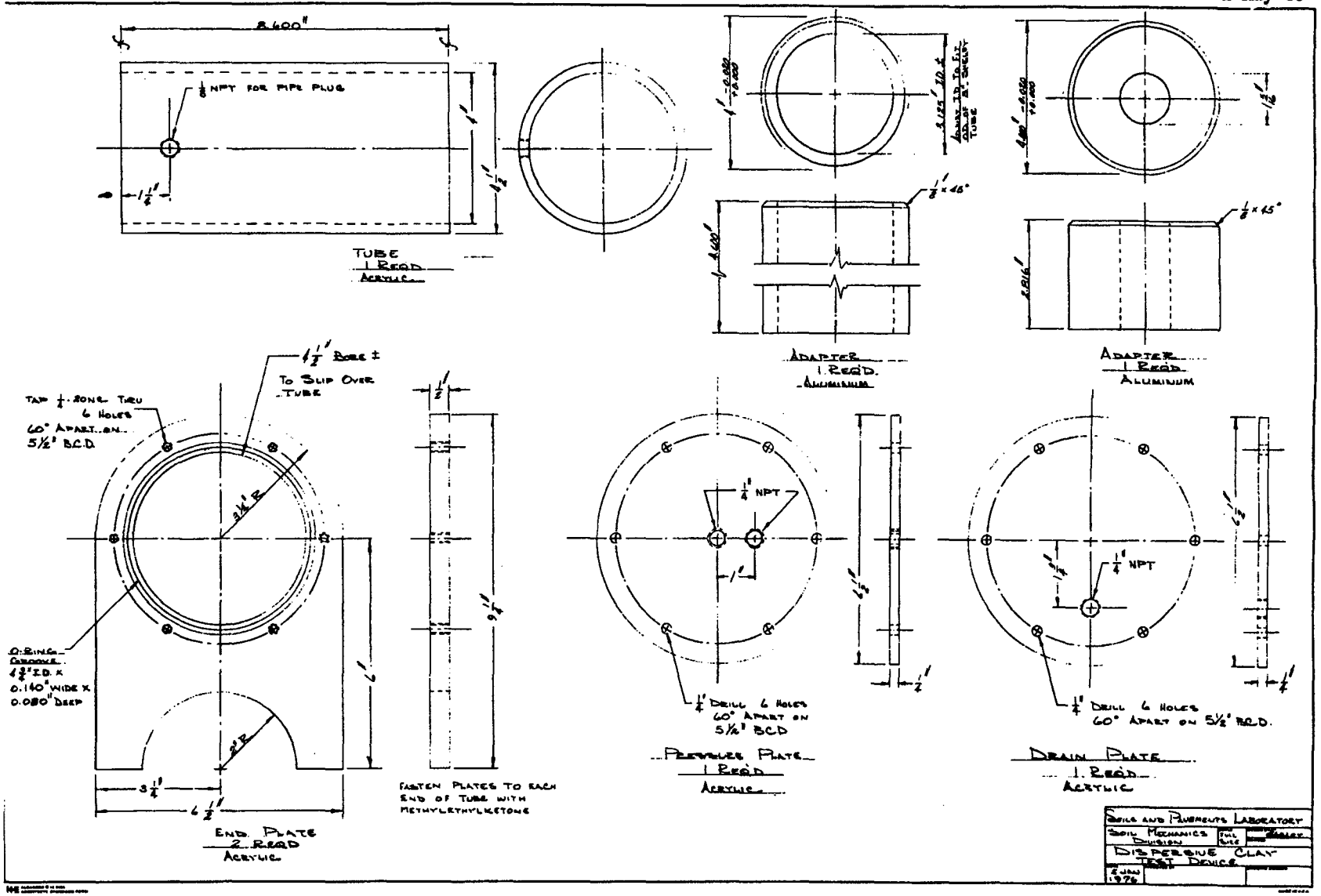

8.400"
⅛ NPT FOR PIPE PLUG
1½"
1"
1½"
TUBE
1 REQD
ACRYLIC
1" +0.000 -0.002
1" +0.000 -0.002
3.125" TO FIT ID OF ⅛" SMOOTH TUBE
⅛ x 45°
4.400"
ADAPTER
1 REQD.
ALUMINUM
4.000" +0.000 -0.002
1½"
⅛ x 45°
0.816"
ADAPTER
1 REQD
ALUMINUM
1½" BORE ±
TO SLIP OVER TUBE
TAP ¼-20 NC THRU 6 HOLES 60° APART ON 5½" B.C.D.
2¼" R
O-RING GROOVE 4½" I.D. X 0.140" WIDE X 0.080" DEEP
½"
9¼"
1"
3¼"
6½"
END PLATE
2 REQD
ACRYLIC
FASTEN PLATES TO EACH END OF TUBE WITH METHYLETHYLKETONE
¼" NPT
1"
¼" DRILL 6 HOLES 60° APART ON 5½" B.C.D
PERVIOUS PLATE
1 REQD
ACRYLIC
1½"
½"
¼" NPT
1½"
¼" DRILL 6 HOLES 60° APART ON 5½" B.C.D.
DRAIN PLATE
1 REQD
ACRYLIC
1½"
¼"
SOILS AND PAVEMENTS LABORATORY
SOIL MECHANICS DIVISION
DISPERSIVE CLAY TEST DEVICE
6 JUNE 1979

PLATE XIII-3

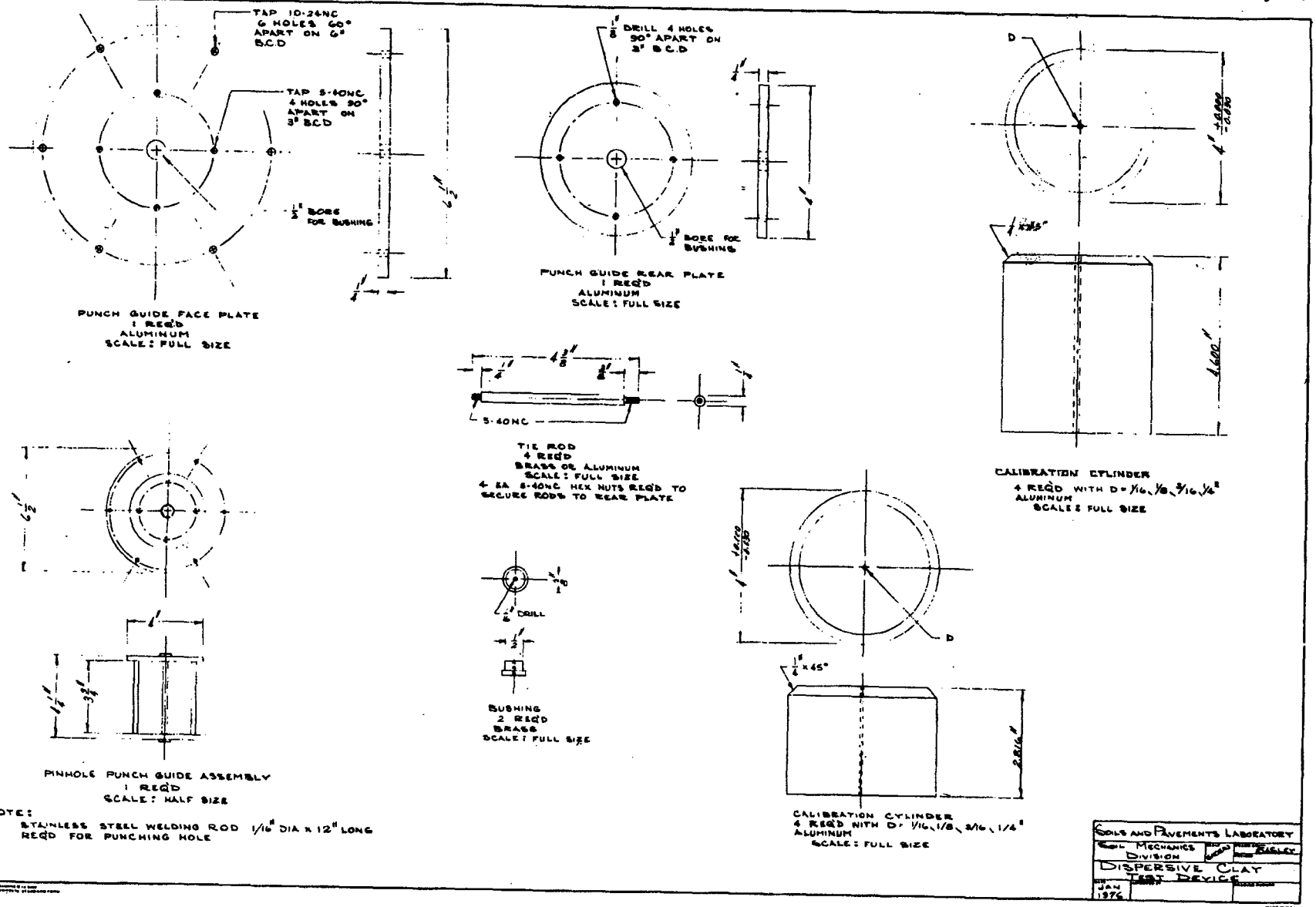
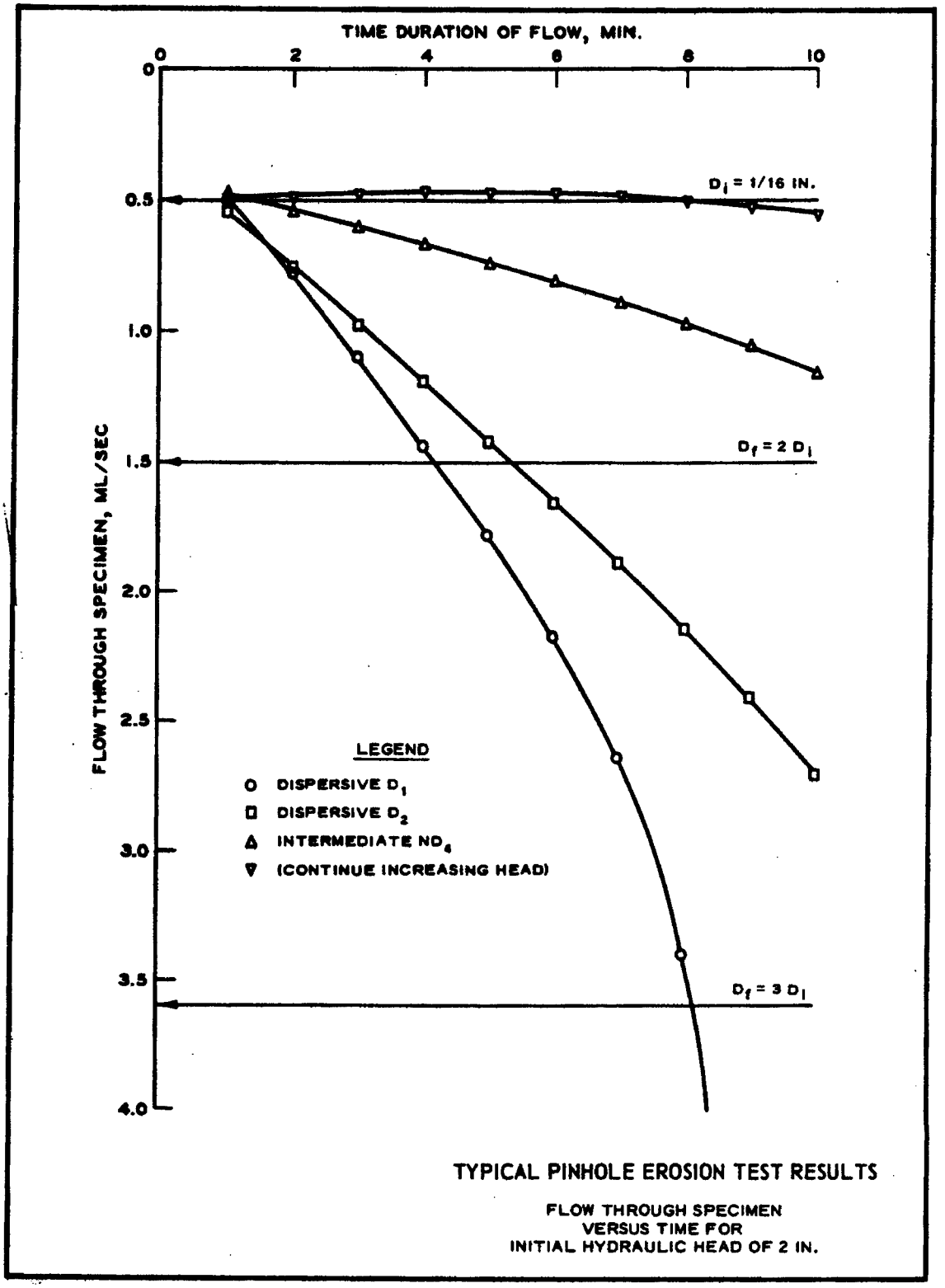

TAP 10-24NC
6 HOLES 60° APART ON 6" B.C.D
TAP 5-40NC 4 HOLES 90° APART ON 3" B.C.D
1/2" BORE FOR BUSHING
PUNCH GUIDE FACE PLATE
1 REQ'D
ALUMINUM
SCALE: FULL SIZE
1/8" DRILL 4 HOLES 90° APART ON 3" B.C.D
1/2" BORE FOR BUSHING
PUNCH GUIDE REAR PLATE
1 REQ'D
ALUMINUM
SCALE: FULL SIZE
5-40NC
TIE ROD
4 REQ'D
BRASS OR ALUMINUM
SCALE: FULL SIZE
4 EA 5-40NC HEX NUTS REQ'D TO SECURE RODS TO REAR PLATE
PINHOLE PUNCH GUIDE ASSEMBLY
1 REQ'D
SCALE: HALF SIZE
DRILL
BUSHING
2 REQ'D
BRASS
SCALE: FULL SIZE
D
CALIBRATION CYLINDER
4 REQ'D WITH D= 1/16, 1/8, 3/16, 1/4"
ALUMINUM
SCALE: FULL SIZE
D
CALIBRATION CYLINDER
4 REQ'D WITH D= 1/16, 1/8, 3/16, 1/4"
ALUMINUM
SCALE: FULL SIZE
NOTE:
STAINLESS STEEL WELDING ROD 1/16" DIA x 12" LONG REQ'D FOR PUNCHING HOLE
SOILS AND PAVEMENTS LABORATORY
SOIL MECHANICS DIVISION
DRAWN
TRACED BAGLEY
DISPERSIVE CLAY TEST DEVICE
DATE JAN 1976

PLATE XIII-4

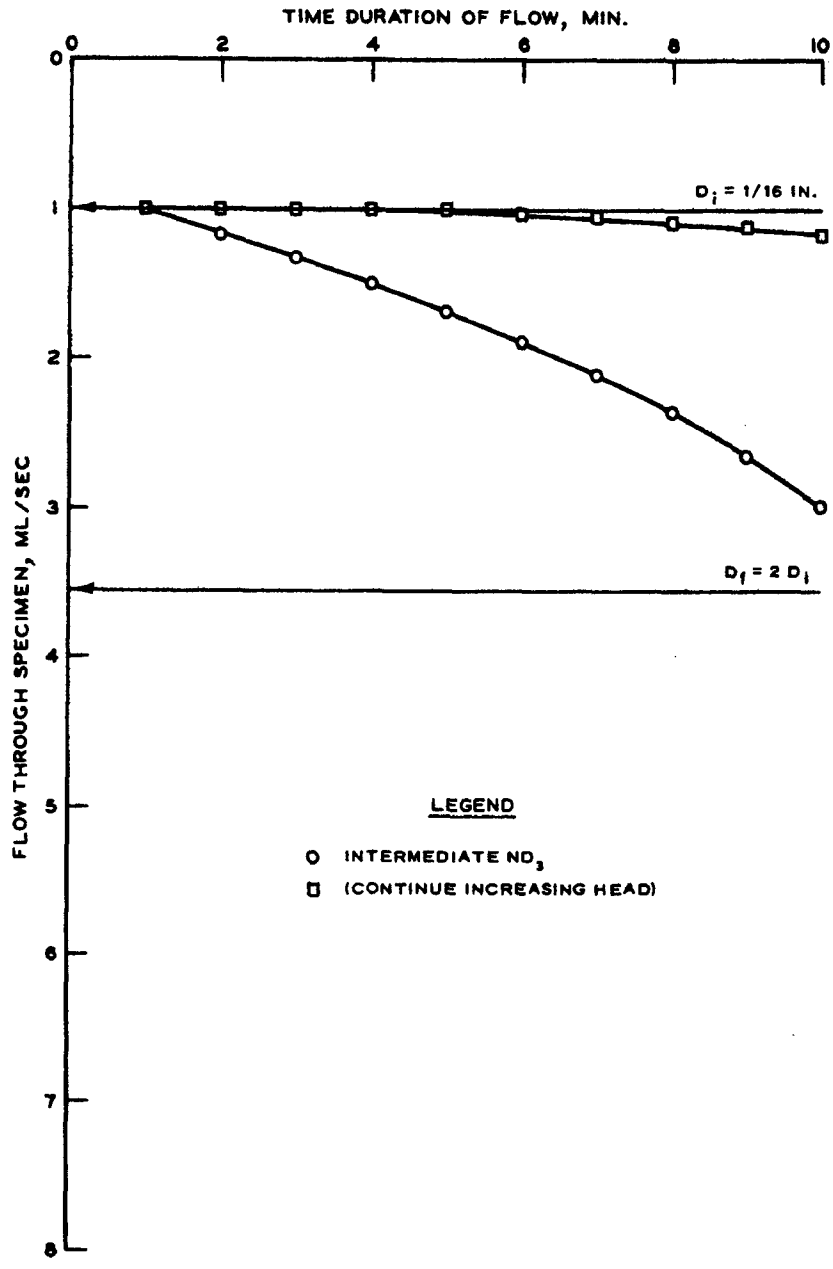
PINHOLE EROSION TEST							DATE
PROJECT							SAMPLE NO.
TYPE OF SPECIMEN: ☐ UNDISTURBED ☐ COMPACTED				CURING TIME FOR COMPACTED SPECIMEN			
ERODING FLUID				TEMPERATURE: BEFORE TEST		END OF TEST	
TECHNICIAN							
TIME	HEAD IN.	FLOW THROUGH SPECIMEN		COLOR OF FLOW			REMARKS
		ML	SEC	CLEAR	CLOUDY	DARK	



TYPICAL PINHOLE EROSION TEST RESULTS

FLOW THROUGH SPECIMEN
VERSUS TIME FOR
INITIAL HYDRAULIC HEAD OF 2 IN.

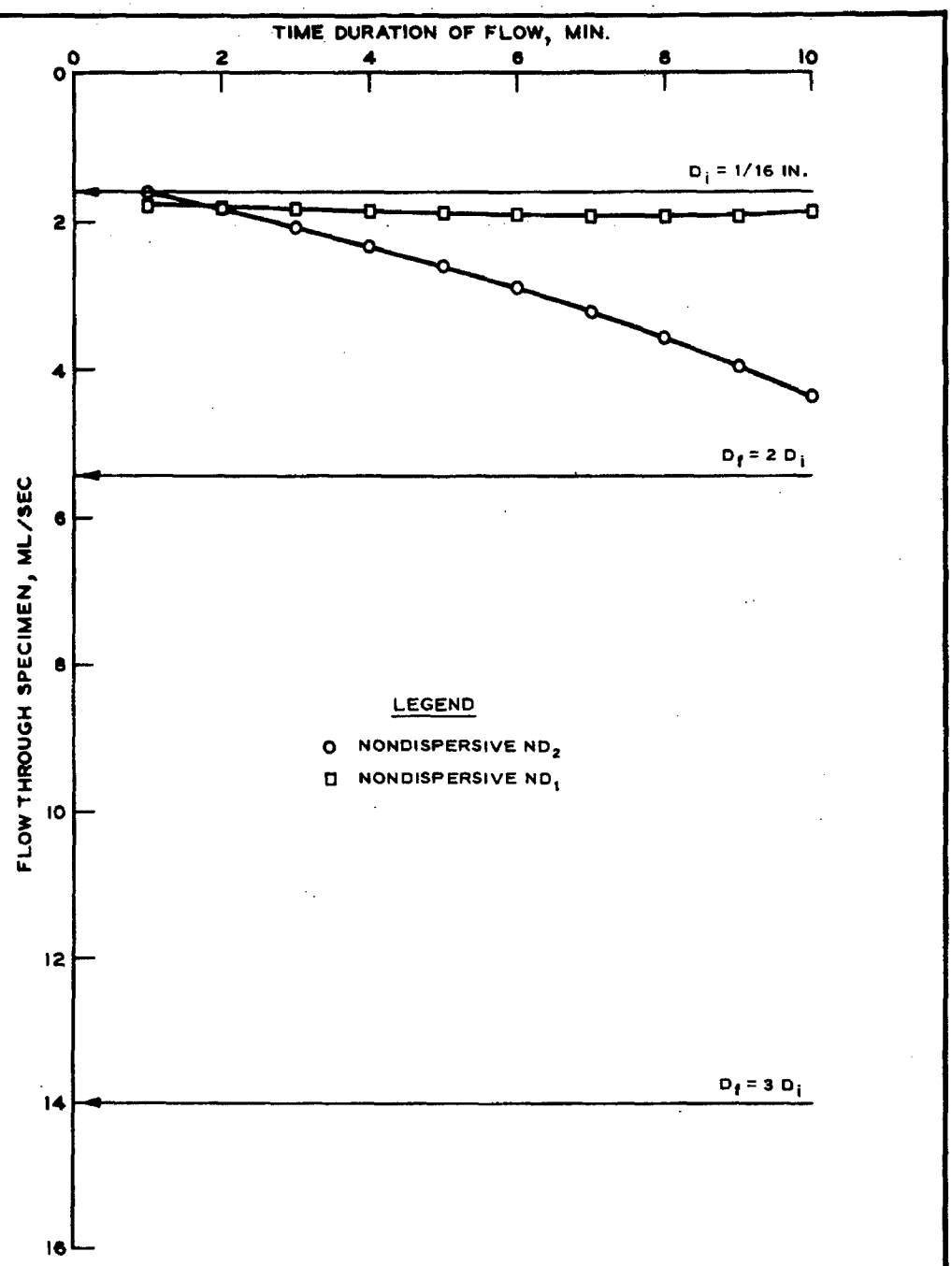
PLATE XIII-6



TYPICAL PINHOLE EROSION TEST RESULTS

FLOW THROUGH SPECIMEN
VERSUS TIME FOR
INITIAL HYDRAULIC HEAD OF 7 IN.

PLATE XIII-7



TYPICAL PINHOLE EROSION TEST RESULTS

FLOW THROUGH SPECIMEN
VERSUS TIME FOR
INITIAL HYDRAULIC HEAD OF 15 IN.

PLATE XIII-8

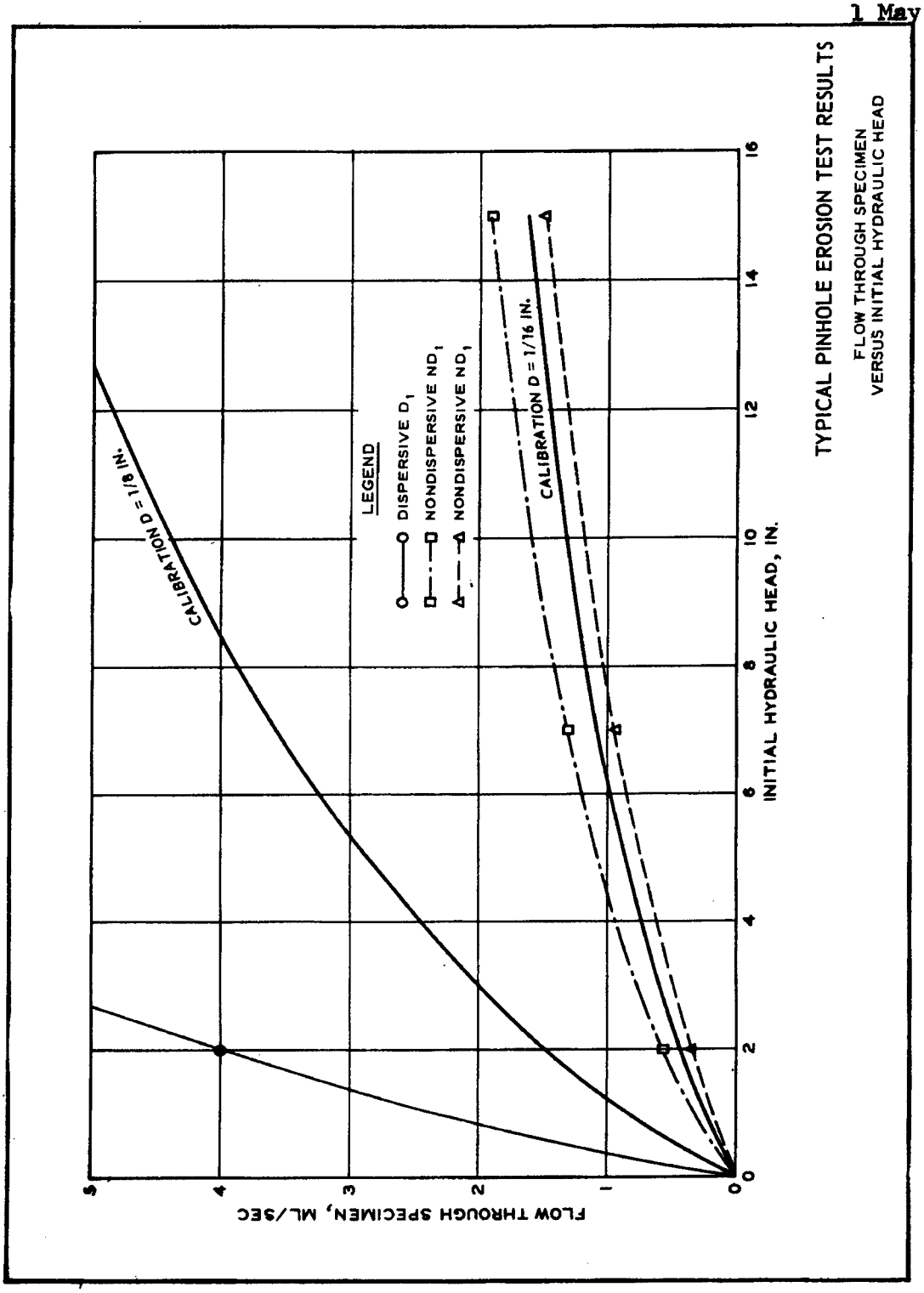


PLATE XIII-9

*U.S. GOVERNMENT PRINTING OFFICE: 1991--524-634/40509